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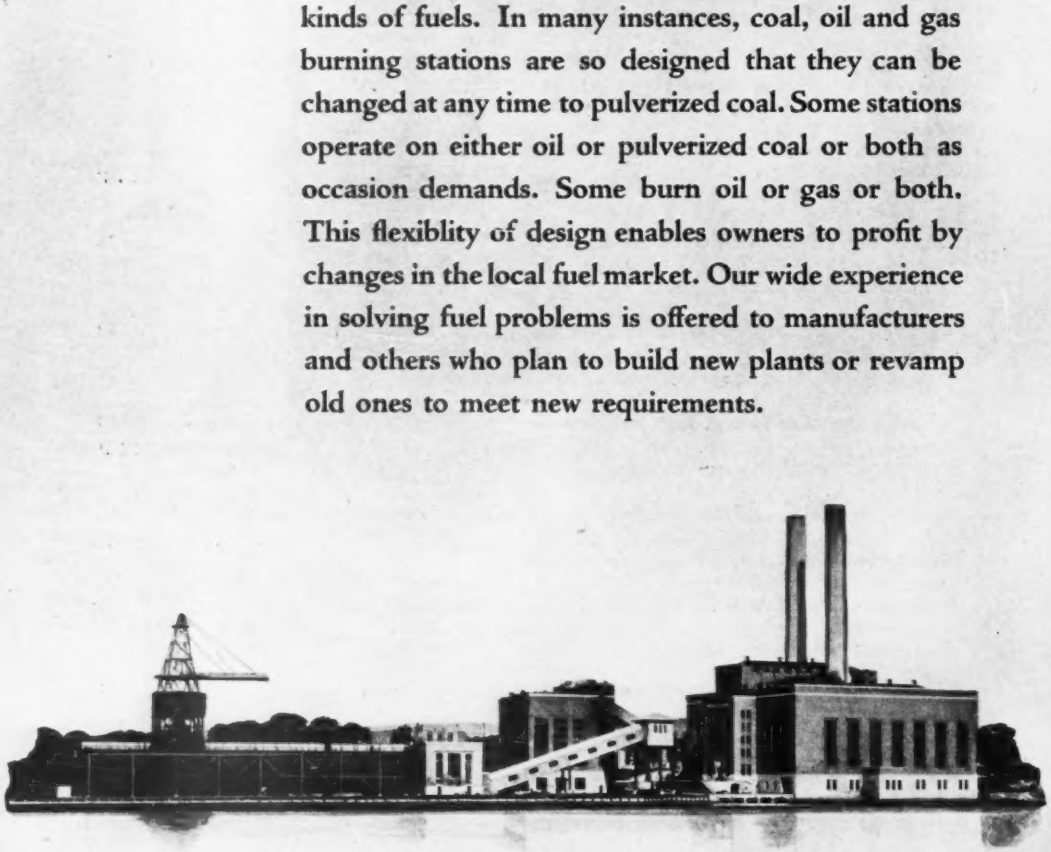
SOLVING FUEL PROBLEMS

STEAM power station work of the Stone & Webster Engineering Corporation includes industrial and utility plants operating with record economy on all kinds of fuels. In many instances, coal, oil and gas burning stations are so designed that they can be changed at any time to pulverized coal. Some stations operate on either oil or pulverized coal or both as occasion demands. Some burn oil or gas or both. This flexiblity of design enables owners to profit by changes in the local fuel market. Our wide experience in solving fuel problems is offered to manufacturers and others who plan to build new plants or revamp old ones to meet new requirements.



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DEVOTED TO CIVIL ENGINEERING AND CONTRACTING

F. E. SCHMITT, *Editor*

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Construction Safety

A WELL-BALANCED program that deserves the attention of contractors and all others interested in the promotion of construction safety will be presented by the construction section of the National Safety Council at the safety congress to be held in Chicago Sept. 30 to Oct. 4. In the past the need for safety measures found understanding among contractors only slowly, but by this time it has made a strong impress on the consciousness of the entire industry. If this gain is to be turned to practical account, however, more attention must be paid to the details and workings of safety management. An example of this need can be seen on almost any construction job in the inconsistency of safety measures. Much attention will be paid to safeguarding part of the work, while equally hazardous conditions in another part are ignored. At the congress the idea of maintained safety will be emphasized. The construction safety movement is making headway, but if it is to continue its advance it will do so only by virtue of eternal vigilance on the part of all concerned. The congress can give the impetus for a fresh start toward greater achievement in safety work.

Handling Dry Concrete

COMMITMENT to the use of dry concrete led to the employment of unusual plant and methods in building the Diablo dam. The specifications required the concrete to be placed either by dump buckets or by belt conveyors, and the unique combination of hoist tower and swinging belt conveyors was the constructor's device to meet the prescriptions of the engineer. Virtually he substituted for the chute lines in the conventional hoist-tower and gravity-chute outfit a set of boom-carried belts. The success of this unit rests largely in the ingenious mechanical control of belt-boom movement by which the discharging stream of concrete can be spread evenly in thin layers over wide placement areas. It is this thin spreading and such impaction as results in dropping the concrete from the belt end upon which the engineers depend for compaction. There is no tamping or handling of the concrete after it is spread by the conveyor beyond some slight shoveling and tramping to bury large projecting stones. The fact that, with no greater effort to consolidate the concrete, material of excellent strength and density is being obtained constitutes an outstanding feature of the Diablo dam work. It indicates that remarkable workability is being secured by the careful grading and proportioning of the aggregates. Despite this success it may ultimately be found that the return to dry concrete now being witnessed in several large dam building operations calls for a corresponding return to mechanical compaction—a modern counterpart of the hand ramming of earlier days. In some dam construction in the East, with a drier concrete, electrical tamping machines are being used, and it may be expected generally that positive compaction will be essential in the use of dry mixtures if the strength and solidity which they promise is to be secured.

Study of Floor Construction

PROPOSED advocacy of the use of steel for building floors is interesting news. The initial design, described elsewhere in this issue, is simple and ingenious, but its real significance lies in the possibility that the efforts necessary for its promotion may lift the whole subject of building floors from undeserved obscurity and subject it to fuller analysis. Some tendency toward such analysis appeared several years ago, when light joists were introduced for floors, but no substantial results were recorded. A new opportunity is now offered for careful study of the problem. The first and most important point to be considered involves the fundamental question of what a floor is supposed to do. Is it merely a platform to carry dead and live loads or should it, in addition, be considered to be a stiffening diaphragm that contributes toward the structure's resistance to wind? Experience in the Florida hurricane of 1926 demonstrated that floors do act in the latter capacity, whether they are designed for it or not. Hence the necessity for a careful scrutiny of floor action to see whether the basic assumptions are adequate. With this question actively under discussion, perhaps the importance of the floor as an integral part of a building frame will become widely enough appreciated that new and continuing studies of its possibilities may be engendered.

Road Money

THE annual highway bill of the United States is commonly assumed to approximate a billion and a third dollars. Probably it is greater; the reports of expenditures on local roads are too poorly compiled and too incompletely furnished by counties and townships to permit of accurate summation. But it is known somewhat certainly what federal and state road income and expenditures are, and these count up close to a billion dollars. The exact figures for 1928 are given in our news pages this week. The broad outstanding fact is that the states, with all their open-handed generosity with highway money, are spending less than their income. At the beginning of the present year the accumulated balance amounted to well over two hundred millions. A second fact is that of the entire state income for roads, motor vehicle fees and gasoline taxes represent 58 per cent—and this in a year when two great states, Illinois and New York, had no gasoline tax and before nearly a score of states had raised their gas-tax rates. Next came receipts from the sale of notes and bonds and from federal aid, 23½ per cent. There remains 18½ per cent coming from other sources, but only one-fourteenth of this is direct state road tax. If, as it is commonly argued,

ideal financing is to build and maintain main roads with "user taxes" and local roads with direct taxes and benefit assessments, it appears from these figures that the ideal has been very nearly attained. In this fortunate condition, highway financing is now beginning to assume trends which call for earnest consideration and appraisal by tax authorities and legislatures. Highway expenditures claim the serious study of everyone interested in the roadbuilding industry.

New Knowledge of Arches

KNOWLEDGE of masonry arches has made long and slow progress since the days of the conventional pressure-line analysis. These latter survived almost until twenty years ago, in spite of the proof of elastic behavior given by the Austrian tests in the nineties; however, within the two decades the practice of elastic analysis established itself quite thoroughly. Now, within the last two or three years, surprising amendments have been made to the basic concepts of that practice, and both arch analysis and arch design are in a distinctly new day. The latest additions, supplied by S. B. Slack's observations on a Georgia arch bridge during construction, are quite as important as those that precede.

In sequence, the developments of recent years may be described thus: First came the recognition that the deformability of piers and abutments affects the strength of the arch, certainly where the piers are high and thin, and probably in all cases; experiment verified the belief, and showed also that footing yield may often be a factor. Second, the conventional division of an arch into barrel (or rib) and superstructure was subjected to question. The participation of solid spandrel walls in the resisting action of the arch was perceived rather early, and was put to practical use in several structures, most notably in the large arch bridge over the Tiber at Rome. Much later the influence of superstructure on the ribs of open-spandrel arches was studied, and elaborate experiments and field tests showed that here also the superstructure is integral with the arch, and that neglect to consider this integrity in the design is both uneconomical and dangerous. Third, there came frequent question concerning the chemical or setting deformations in concrete arches, and doubts as to the magnitude of temperature stresses. This last group of questions comes to culmination in Mr. Slack's investigation, whose result is to upset practice in one important point and broaden the designer's outlook considerably in another.

So clearly does the Georgia work show the early genesis of arch resistance in the structure as built on its centering—as early as the fourth day after keying up, Mr. Slack reports—that it is seen that the subsequent temperature stresses occurring in service must be those corresponding to fall of temperature only, and therefore will be twice as large as usually computed. The ratio may even be greater if the setting temperature in the concrete goes above maximum summer temperature. Flat arches are seriously affected by this discovery, since they have relatively large temperature stresses. A further increase in the tax on the arch may be caused by the shrinkage and cooling of the superstructure concrete, which is usually placed at a time when the arch has already cooled. But if construction methods can be so arranged as to reduce the abnormal temperature effect—by jacking and late keying, perhaps—abnormal temperature stress may be minimized and distinct economies realized.

These important modifications of current views and practices are supplemented by the curious divergences from calculated stress in both concrete and steel which the Georgia results display. Measurements on several test blocks indicate that these are probably due in the main to chemical changes in the concrete, and may therefore be called chemical stresses. They are not fully clarified by the present data, but enough is shown concerning their nature and magnitude to give them claim to careful consideration in the estimation of arch strength.

It is necessary to keep in mind the findings of the North Carolina ultimate-load test on an arch bridge, while considering the Georgia results. The North Carolina work, aside from its great importance as a full-scale demonstration of the integral nature of superstructure and arch, showed that an adequately designed modern reinforced-concrete arch has a surprising reserve of strength, and that even under failure load it does not collapse. This is true in spite of the extra stress elements revealed by Mr. Slack's observations. Hence it is fair to conclude that this latest work does not give ground for alarm concerning the safety of well-designed structures. But it does help to explain why some arch bridges, built much lighter than normal practice because of their designers' faith in the excess strength which concrete structures are apt to display, have ultimately failed, rather disconcertingly. In short, the new results lay emphasis on the value of conservatism in arch design, and upon the need for skilled and thorough analysis before fixing the proportions of a proposed structure. Rule-of-thumb practice in arch design is definitely obsolete.

More investigational work needs to be done in order to confirm and complete the showing of the Georgia studies. Their very important conclusion as to temperature stress should be checked as soon as may be possible by equally careful work on another bridge during construction. Additional observations on strains in concrete and steel from the true no-stress state also need to be made, to round out and perhaps to explain the unexpected divergences shown. Even without this, however, the results take an important place in the modern knowledge of arches.

One special deduction from the new work is of further interest. The demonstration of integral action of superstructure and rib, obtained through the North Carolina measurements, Beggs' investigations on celluloid models of arches, and Wilson's measurements on large laboratory arches, had one peculiar feature. Professor Wilson's work revealed that in helping to stiffen the arch rib the superstructure may become overstressed before the rib itself is endangered. In short, it became evident that the usual forms of superstructure are inefficient devices for stiffening the arch rib; and at the same time the utilization of the superstructure for this purpose was all the more to be questioned because stiffness is not wholly a virtue in arches, in view of the major importance of temperature stresses in many arches—and temperature stresses increase with stiffness. But since it is now shown that temperature effects are much greater than has heretofore been believed, it follows that temperature stresses will be controlling even more frequently. Accordingly, the co-operation of superstructure with rib may be objectionable rather than desirable in many cases; and if so, it will become the designer's task to develop means for separating superstructure and arch rather than combining them. Providing for this independence and taking care of the always

troublesome temperature expansion in the deck should offer many interesting opportunities to the designer.

The preceding refers to reinforced-concrete arches. Stone masonry arches attained a high place long before reinforced concrete entered their field, and their successful survival testifies to the value of what was accomplished by the simple pressure-line analysis. The old theory need not be totally forgotten, therefore, for its fault is incompleteness rather than untruth. Moreover, the engineer will not overlook the essential differences in structural character that distinguish the reinforced-concrete arch from the masonry prototype. The latter had characteristic virtues, and the greater possibilities of the concrete arch should not cause them to be forgotten.

County Engineering

ELEVATION of the county engineer's office has come with activity in highway improvement. There are many counties whose engineers carry on roadwork having a volume and value equaled ten years ago in only a few states. Their construction and maintenance organization and equipment and their engineering technique vie with those of the best state highway departments. These examples, however, are less significant of county engineering advance than the evidence that counties everywhere, even to the most remote of all the 3,074 in the United States, are on a higher plane of engineering conduct of public works because of roadbuilding. It is this situation with its implications of promise and hazard that has led to the article in this issue on Caddo Parish engineering and will bring other articles from time to time on county engineering practice.

There is need that the engineering profession at large should understand the vital importance of high standards in county engineering. The county engineer is the last line of defense of efficient roadbuilding. The great secondary road system of the country is his charge, and no prospective advance in centralization of roadbuilding authority is likely to take it from him if he fulfills his function. It may, however, very possibly bring the township road under county direction, and increase the county engineer's responsibility to embrace nine-tenths of the country's road mileage. There must be high standards of engineering if the task is adequately performed, and unless it is so performed further delocalization of highway control is the certain outcome.

The advance to present partial state and federal control of highways has drawn its power largely from the inability evidenced by the smaller units of government to cope with the technical problems of the modern highway. The township is too small a political unit to hold out inducement to capable road engineering. The county can do so. If it does not, it is in line to be eliminated in the scheme of road administration and have its duties assigned to state and federal authority in order to secure the efficiency and economy which modern highway transportation will not forego.

As yet there is little or no recognition of the county engineer, such as is accorded the city engineer. There are few signs that technical societies are helping the county engineer or understand his problems. But perhaps county engineers can help themselves. As a beginning, greater consciousness of county engineering is needed; increased solidarity and efficiency of this important body of engineers in public works will naturally follow.

Dynamic Analysis

WHOLLY new matter is presented in the discussion of methods of analyzing the counterweight system of the ill-fated Hackensack bascule bridge, in this issue. The discussion is a fundamenal contribution to structural literature, because the analysis of these essential auxiliary mechanisms of movable bridges has not previously been recorded in public form. This is partly true even of static stresses, and entirely true of the dynamic stresses which are now under consideration.

Loaded-spring action is not commonly part of the structural engineer's concern. In the analysis of the Hackensack counterweight system in the investigations following the failure of the bridge a few months ago, such action occupies the central place. While the mechanics and study of such action are well established, their application to a linkage of the kind here involved, and to great moving masses, invites careful study.

In its general bearing on structural engineering, the subject of discussion is important because designers have always made rigidity a prime objective. It is true that they recognized every structure to be elastic, or flexible; yet they endeavored, consciously or unconsciously, to limit the degree of flexibility to such small proportions as to make it negligible. This is one reason, among others, for the avoidance of flat lattice angles and bad systems of bracing. It is also a reason for the tendency in both bridge and mill building design away from "soft" or flexible forms of structure and toward highly rigid forms. In another field, that of suspension bridges, the necessity for minimizing flexibility has been well appreciated since an early day, and the dangerous possibilities of cumulative sway are duly considered, even though empirically.

In the present case we have a huge counterweight mass poised above a public roadway and held from tipping by a rather flexible and springy tower post. Such a condition not only is vulnerable to any abnormal action but draws very urgent attention to the question of possible resonant oscillation in the elastic system composed of the mass and the spring. It was found by observation that this system can experience a horizontal range of vibration of many inches, confirming calculation results. A vibration of this kind clearly becomes a matter of utmost importance with respect to the service and safety of the structure.

Ordinarily one is apt to depend on friction damping to check or control such oscillations. It appears, however, that under certain conditions friction damping does not act. With damping eliminated, the possibilities of resonant vibration become so menacing that conservative engineering will not countenance the danger. It is a plain question whether the design of structural assemblages under these circumstances may be permitted to include elements of large flexibility. Rigid bracing is a vital safeguard.

It should not be assumed that the analyses now presented include all the problems arising in the mechanism of such a bascule bridge as is discussed. The analyses were controlled, as will be plain on reading them, by the phenomena of the accident which wrecked the bridge some months ago. Numerous other problems of mechanism and structure reside in this bridge as well as in other bascule bridges, and some of them may yet prove to be commensurate in importance with the specific one now discussed, in their bearing on both specification and design. The present contribution of thought makes a significant beginning in the discussion of movable bridge analysis.

Construction Ingenuity Displayed at Diablo Dam

Contractor on Seattle's 425-Ft. Dam on the Skagit River Exhibits Notable Genius in Devising Methods for Placing Dry Concrete and Sealing Foundation Site by Grouting Transverse Zone of Understream Strata

BELT conveyors swung from hoist towers in the manner of gravity chutes are placing a dry, lean mix and securing remarkable concrete strengths in building the Diablo dam on the Skagit River in Washington. Besides this wholly unique device for handling the concrete, exceptional use was made of grouting to form subsurface cofferdam walls in the permeable boulder-gravel bottom of the river so that the dam foundation could be laid dry for excavation. Combined with the design of the dam as a constant-angle arch between gravity-section abutments and the remarkable height of 425 ft., these construction features make the Skagit River operation one of the most unusual in modern dam building.

Location and Structure—Diablo dam is being built on the Skagit River in Washington as part of a comprehensive water-power development on that stream which is being carried out by the city of Seattle. The present project is about 100 miles northeast of Seattle, or 10 miles upstream from the city's Gorge Creek power plant, some of whose features were described in *Engineering News-Record*, Nov. 18, 1920, p. 994, and Oct. 30, 1924, p. 700. Primarily the purpose of the Diablo dam is to store water for power to be produced in a plant which will form part of the present project, but the storage will also be useful to the plant below. A third project, scheduled for construction by the city at a later date, is located still farther up the river.

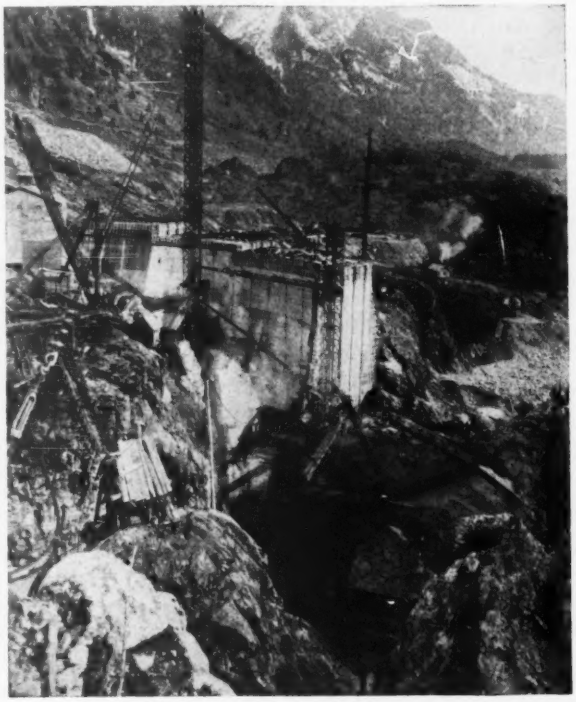


FIG. 1—GENERAL VIEW OF DIABLO DAM FROM THE LEFT BANK

Note belt conveyors depositing concrete. Also foundation for third location of steel tower on downstream face of dam.

The site of the dam is in a narrow gorge whose solid granite walls rise perpendicularly for nearly 200 ft. above river level and in places actually overhang. Sinuous windings and varying widths make the gorge picturesque; at the dam site the canyon is only 125 ft. wide at water level, and there is a point only a few hundred feet below where the sheer walls are not more than 15 ft. apart.

Structurally the central part of the dam is an arch (designed by the Constant Angle Arch Dam Company). Both ends of the arch, however, terminate in full gravity sections which contain the spillways. The reservoir will have a capacity of 90,000 acre-ft. and a drainage basin of 1,100 square miles, part of which extends into Canada. Of the 316,000 cu.yd. of concrete to go into the dam under the present contract, one-half had been placed on June 15.

Stream Control and Grouted Cutoffs—Diversion of streamflow to permit unwatering the dam foundation was effected by driving a tunnel 20 ft. in diameter and 657 ft. long through the solid granite around the south abutment. Under the 8-ft. head produced by a timber crib diversion dam faced with steel sheetpiling, the tunnel capacity is 7,000 sec.-ft. The crib dam was built up of fir logs cut from the near-by hillside, many of them 3 or 4 ft. in diameter. Two floods of 22,000 and 26,000 sec.-ft. passed through and over the diversion works during construction, but though some of the cribbing and rockfill was lost, the project was not seriously delayed. The diversion tunnel was started Dec. 28, 1927, and it was put into service April 11, 1928. Ultimately the 75-ft. length of this tunnel directly under the main concrete dam is to be filled with a concrete plug.

Large boulders in the streambed complicated the problem of building the cutoffs to bedrock above and below the main structure preliminary to unwatering the site. Their presence precluded sealing by steel sheetpiles. The method used was to drill wells down to the solid rock and fill the spaces between boulders with grout forced in under pressure. Six-inch holes were drilled 9 ft. apart in two rows across the gorge, staggered. The greatest depth from streambed to bedrock was 50 ft. Grouting was begun at the bottom and the grout pipes were withdrawn gradually as a seal was effected.

This grouting was necessarily a blind operation, but it is believed that it built up a wall 25 ft. thick extending across the bed of the gorge. Leakage observed later indicated that the only deficiency of the grout wall was due to the impossibility of getting the grout behind some of the flat boulders which lined the flanks of the gorge.

The downstream cutoff structure was located about 100 ft. below the toe of the main dam at a point where the gorge was about 60 ft. wide. The upstream cutoff

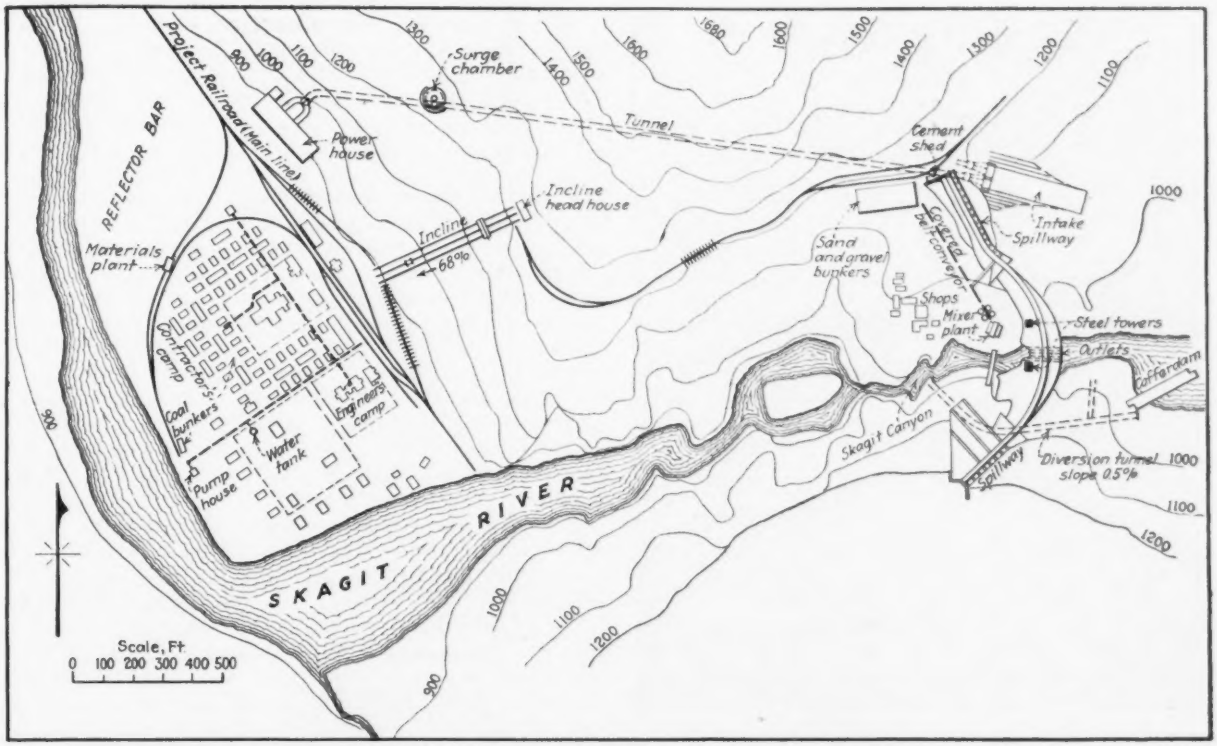


FIG. 2—VICINITY MAP SHOWING CONSTRUCTION PLANT ARRANGEMENT AT DIABLO DAM

was immediately upstream of the heel of the main dam.

Excavating the Foundation—Stripping the canyon walls included the removal of all rock about which there was any question as to its solidity and as to its being an integral part of the underlying formation. Diablo Gorge is cut through very hard, fine-grained granite and granitic gneiss having a crushing strength of 25,000 to 28,000 lb. The stripping was begun before the streambed was unwatered, the idea being that high water would aid in carrying away the débris. While the work was in progress and during spring flood stages, a dragline bucket was operated in the gorge, dragging downstream between the cutoff structures with a view to dislodging material and aiding the current in carrying it out so as to reduce the yardage to be excavated after diversion.

Streambed excavation with power shovels was begun soon after diversion. The work became extremely difficult when the shovels got down to the firmly embedded boulders of the lower streambed. Blasting was not very effective, and yet the boulders were so well compacted in the mass that manganese dipper teeth seldom lasted more than a shift. Seepage into the excavation reached a maximum of about 20,000 gal. per minute. The heavier pumping installation was near the upstream cutoff structure. Here water was pumped out in two stages of 45 ft. each, using an intermediate flume and sump.

Materials Supply and Control—Because of the river topography (a deep, narrow and sinuous gorge), the materials plant was located at Reflector Bar, about 2,000 ft. downstream from the dam, where a bend in the river caused the formation of a sand and gravel deposit. Transportation between the bar and the mixing plant on the canyon wall above the dam is provided by

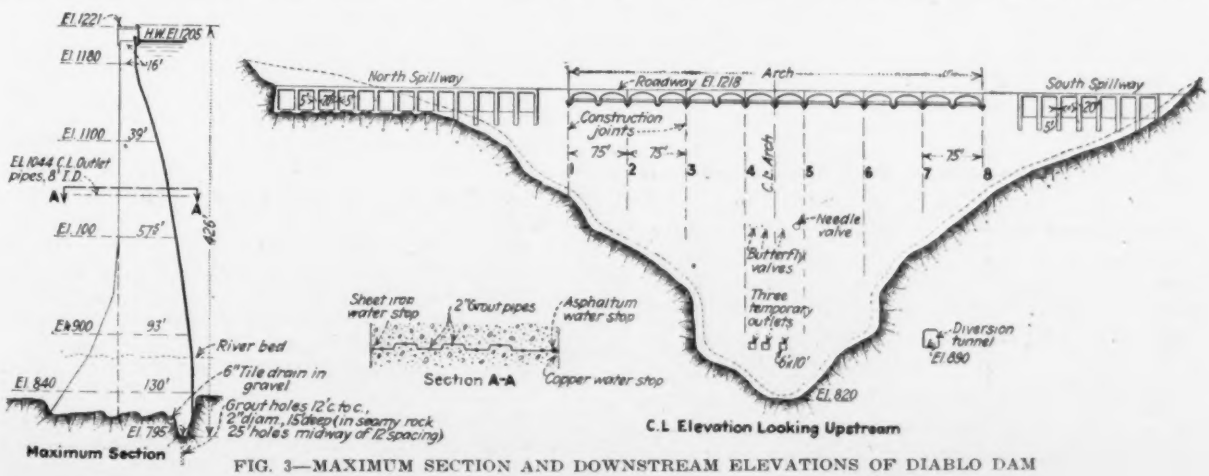


FIG. 3—MAXIMUM SECTION AND DOWNSTREAM ELEVATIONS OF DIABLO DAM

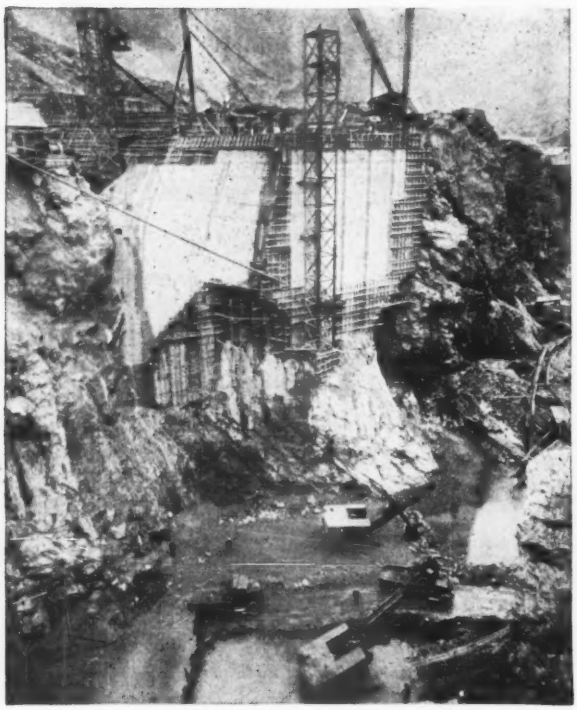


FIG. 4—EXCAVATION UNDER WAY IN BOTTOM OF CHANNEL

Overhung concrete forms made it possible to complete upper section of dam before streambed could be excavated.

a transfer platform and a standard-gage railway built by the city with a view to future use in the development of the project farther up the river. This transfer platform was described in *Engineering News-Record*, July 19, 1928, p. 84.

At Reflector Bar sand and gravel are taken from the riverbed by dragline excavators and conveyed by belts to the screening and washing plant. Gravel up to 4 in. in size is used in the concrete and all aggregate between the 4- and 10-in. sizes is passed through a gyratory crusher. The materials are classified into three grades, sand, medium aggregate and large aggregate.

A high degree of refinement is used in proportioning these materials. Because of this very careful attention to grading a 1:7½ mix by weight has given a strength that would be expected from a 1:5 mix—that is, 28-day compression tests on 6-in. cylinders show an average of 3,978 lb. per sq.in.

After experimenting with the aggregate it was found that concrete with a strength of at least 3,000 lb. in 28 days could be secured with the addition of only 15 per cent more cement than would have been necessary for

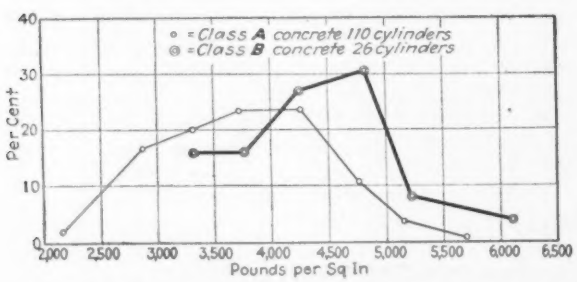


FIG. 5—TESTS ON 28-DAY CONCRETE IN 6x12-IN. CYLINDERS

2,000-lb. concrete. It was therefore decided to provide the additional cement as an aid in securing the impermeable quality so desirable on work of this sort. Strength results for class *A* concrete are plotted on the accompanying diagram; class *B* concrete, used in the 3 ft. next to the upstream face, averaged 4,480 lb. per sq.in. The wide range in the plotted values is ascribed to the fact that at different times cylinders were made by nine individuals. The breaks showing lowest strength were caused by pieces of aggregate 3 to 4½ in. in diameter finding their way into the 6-in. cylinders. Tests for impermeability in 8-in. cylinders made of concrete taken out of the mix as placed on the dam showed no seepage under pressures of 175 lb. per sq.in. maintained for two days.

The fineness modulus method of grading the mix is used, the average moduli of the three grades of materials being 3.3 for sand, 7.6 for medium aggregate and 9.6 for large aggregate. For a 2.1-cu.yd. mixer charge of

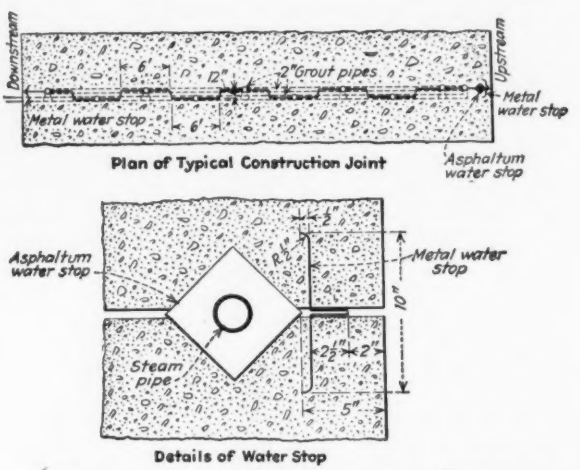


FIG. 6—PLAN OF VERTICAL JOINTS, AND WATER STOP DETAIL

class A concrete, the proportions by weight are cement 1,003 lb., sand 2,470 lb., medium aggregate 3,090 lb. and coarse aggregate 2,080 lb. Altogether this gives the required ratio of 1 lb. of cement to 7.5 lb. of aggregate. (By volume of dry rodded aggregate the ratio would be about 1:5.4.) The sand weight includes an allowance of 120 lb. for water content. This was found to be the average, the range being from 90 to 150 lb. per batch.

The specifications for sand require that not less than 20 and not more than 40 per cent must pass a 30-mesh screen. In sampling sand the requirement is that the 1-gal. sample taken from each 50-ton car shall consist of sand collected in eight separate dips into the stream of material flowing into the car. The car does not pass test unless this sample shows less than 75 per cent by weight retained on a No. 28 screen. If between 76 and 80 per cent is retained on the 28 screen, the car is accepted on the condition that another car of sand having more fines than this minimum limit can be delivered to the storage bins at the same time. If more than 80 per cent of the sample is retained on the 28 screen, the car is rejected unconditionally.

Medium aggregate is required to pass a 2-in. screen and be retained on a ¼-in. screen. Then all pea gravel (¼ to ⅜ in.) is removed from the medium aggregate and just enough of it is put back to bring the fineness modulus within the limits of 7.45 and 7.65. The large aggregate

is required to pass a 4-in. screen and be retained on a 2-in. screen.

Cars of material from the screening plant are dumped into sidehill storage bins above the mixer plant, the total capacity here for all three classes being about 8,000 cu.yd. in three separate compartments. Experience with this sidehill storage showed that when the bins were nearly empty the larger pieces in the two aggregate sizes would roll to the lower ends of the bins, thus giving a separation of sizes. This was remedied by dragline scrapers on the lower ends of the bins, which dragged down incoming material to maintain uniformity. From this aggregate storage each of the three classes of material is delivered separately to bunkers in the mixing plant by a belt conveyor.

Cement is delivered to the job in bulk and is unloaded by buck scrapers operated by a small electric hoist. The cement crew consists of four men, two unloading from the boxcars while two work at the mixer-charging platform. Because the unloading is such disagreeable work, these pairs alternate between the mixer and the car. Double time is paid for unloading from box cars.

A belt conveyor about 700 ft. long delivers cement from the unloading shed to four wood-stave tanks or silos at the mixer plant, which have a combined capacity of about twelve carloads. The bottoms of these tanks, rebuilt to hopper shape, deliver through a screw conveyor to scales immediately over the mixer hopper, where the amount prescribed for each batch is weighed.

At first the cement charge was dumped into the mixer hopper from a small shuttle car. The nuisance and loss resulting from the cement dust made necessary a change to a short screw conveyor, which has been most satisfactory.

Mixing and Placing Concrete—To secure the prescribed slump of not more than 3 in. (averaging between 2½ and 3 in.) a total of 5¼ gal. of water is used per sack of cement. The amount of water added is decreased to allow for the 120 lb. of water contained in each sand charge, and the actual water-cement ratio is approximately 0.70.

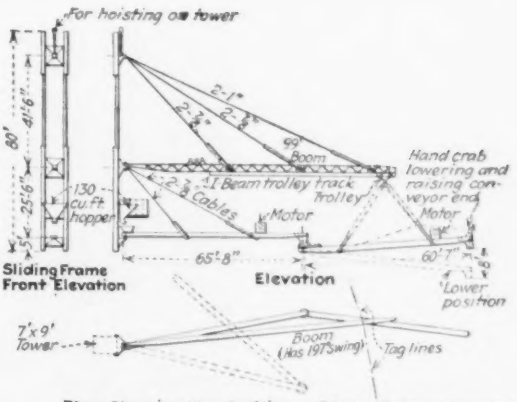


FIG. 8—ARRANGEMENT OF CONCRETE BELT-CONVEYOR SYSTEM

A single weighing device measures the quantity of sand, medium and large aggregate for each batch, three settings of the scale beam being necessary to complete the charge. It was found important to require that the sand be measured as the second instead of the third element of this charge. This arose from the natural tendency to overrun rather than to underrun on the first two weights, compensating for such overage by cutting down on the weight of material admitted in the third measurement. Weighing in the coarse aggregate last obviated any ill effects from this practice.

Unique Belt Placing System—The conveyor system consists of two belts, the inner one slung from the tower and the other suspended from a trolley on a steel boom, as shown in the accompanying drawing. Movements of the two belts are controlled from an operator's cab on the boom, whence the trolley can be moved in and out and the boom can be swung right or left. These operations are effected by three small air hoists. Each belt is driven independently by a 5-hp. motor. A hand crab at the end of the outer conveyor permits adjustment of the discharge level. The entire conveyor system is slung on tackle so it can be hoisted up the tower as the work progresses.

The boom supporting the outer conveyor has a horizontal swing of 197 deg., thus permitting the conveyors it supports to be jack-knifed through an arc of almost 360 deg. The delivery of concrete from one conveyor unit to the other is through a swivel spout which ties the two together. The operation of this connection was not entirely satisfactory and free from slopping until a metal plate on a 45 deg. angle was substituted for a direct drop.

The 18-in. belt originally in-

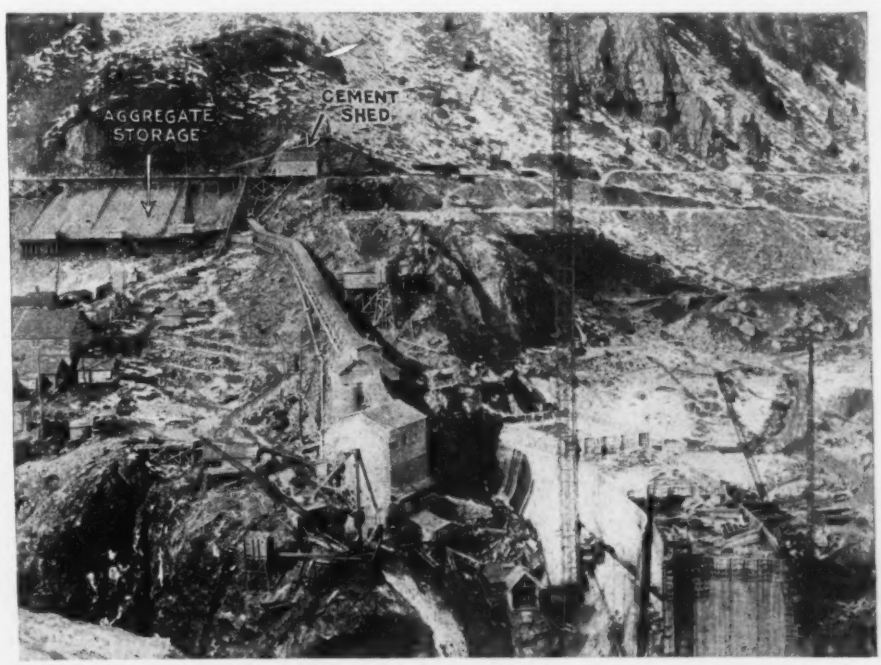


FIG. 7—RIGHT BANK, SHOWING CONSTRUCTION PLANT LAYOUT
Covered belt conveyor operates between cement shed and silos. Aggregate also is delivered by belt.

stalled was too narrow and was replaced by a 24-in. belt, with corresponding reconstruction of the supporting structure.

The inner belt runs at a speed of 290 ft. per minute, the outer belt at 305 ft.; the difference reduces the danger of overloading at the junction point. Similarly the opening through which concrete passes from one belt to the other was made 12 in. in diameter, whereas the opening through which the first belt is loaded is only 10 in. This concentrates any tendency to choke or clog at the point where the belt-loading operator is stationed. The outer end of the belt-conveyor system discharges through an elephant trunk.

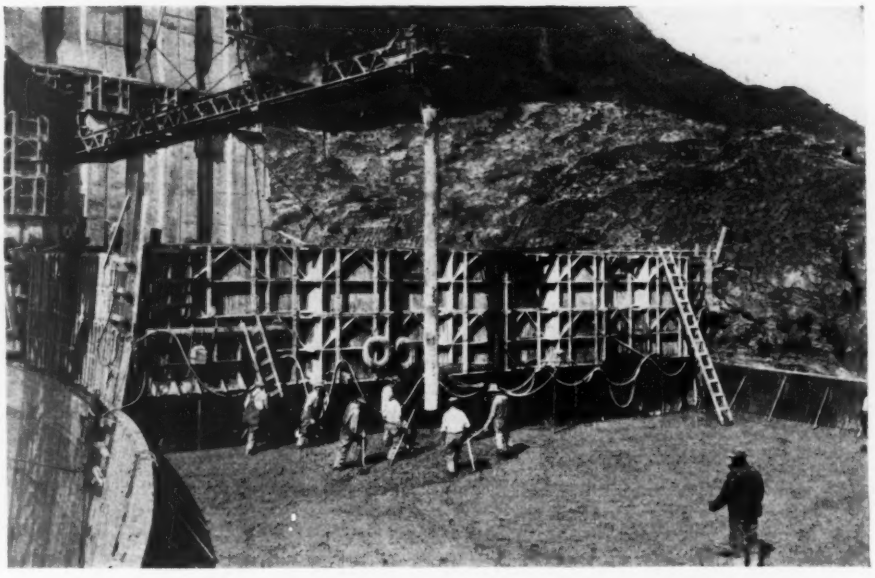


FIG. 10—VIEW OF ELEPHANT TRUNK DEPOSITING CONCRETE

The conveyor crew consists usually of two men, one operating the discharge gate on the hopper at the tower end of the belt and the second riding in the boom cab and moving the conveyors at the signal of the puddling gang foreman. A third man occasionally helps in raising the outer end of the conveyor.

In contrast to delivery systems of less flexible character which involve pyramiding and subsequent spreading by hand, the delivery end of the belt conveyors may be swung slowly and continuously across the surface of the dam while delivery is in progress. Thus a continuous layer of concrete can be placed 6 to 18 in. thick as desired by varying the speed of travel. The men of the puddling gang do not have to swing the elephant trunk. Continuous movement over the entire surface to be concreted can be maintained, and a 5-ft. lift is ordinarily completed in five continuous 1-ft. layers. The concrete is compacted by the drop from the spout. Very little handling is required for final placement. Gravel pockets are distributed by shovels, and rocks which show on the surface are forced down by the laborers with their feet and with shovels. The belt conveyors are regarded as essential factors in the notable uniformity of the concrete and in the achievement of a dry mix placed without impact, although from the contractor's point of view the operation of the special rig and the delays chargeable to the dry mix are a disadvantage.

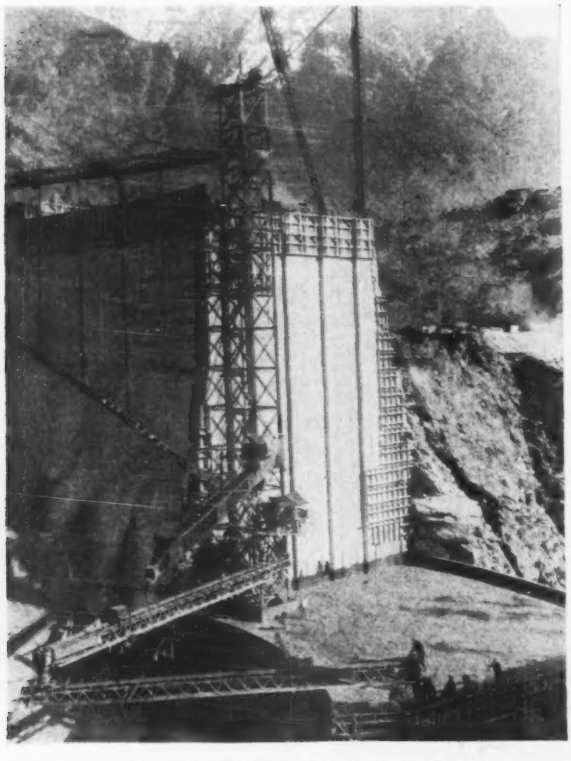


FIG. 9—VIEW OF CONVEYOR SYSTEM SUSPENDED FROM BOOM

By this method of placement concrete is deposited in layers rather than in cones.

The belt conveyors were originally designed to handle 80 cu.yd. of concrete per hour with 18-in. belts. On test runs with 24-in. belts they have delivered 110 cu.yd. per hour and the maximum 24-hour run has been 1,524 cu.yd. The average day's run in the month of June (three shifts) was 1,143 cu.yd. The largest pour in a single month thus far was 30,850 cu.yd., placed in June. (Concrete was poured on only 27 days in that month.)

The forms are stripped after three days and then newly exposed faces are cured under spray for seven days. Horizontal surfaces are brushed as soon as the initial set has taken place, and before the succeeding pour they are hosed off under a pressure of 150 lb. per sq.in. followed by air jets to blow out the pockets.

Directing Personnel—The contract for the construction of the Diablo dam is held by Winston Brothers Company, of Minneapolis, L. S. Oakes, president. W. H. Gardiner is resident manager at Diablo for the contractors, M. S. Glavin is superintendent and F. T. Hillman is construction engineer. Work is being done under the general direction of W. D. Barkhuff, city engineer of Seattle. T. H. Garver is assistant city engineer in charge of water supply and power development. E. D. Alexander is resident engineer for the city at Diablo dam.

Freight Station With Air-Rights Warehouse at Chicago

World's Largest Commercial Building Being Erected Over City Terminal of the Northwestern Railway

AS A PART of the general revision and improvement of its freight terminal facilities in Chicago, the Chicago & Northwestern Railway is building a large freight station for less-than-carload service, covering the site of its old passenger terminal at Wells St. and fronting on the north side of the main stem of the Chicago River. It will have a capacity for handling 10,000 tons of freight daily, as compared with a present capacity of 1,000 tons at the existing two old freight stations in the vicinity. All less-than-carload freight will be handled by trap cars between this city station and the large freight transfer station at the Proviso yard, 12 miles distant. There all inbound freight is transferred to the trap cars, while outbound freight from these cars is consolidated into full carloads for various destinations, as described in *Engineering News-Record*, Feb. 23, 1928, p. 317.

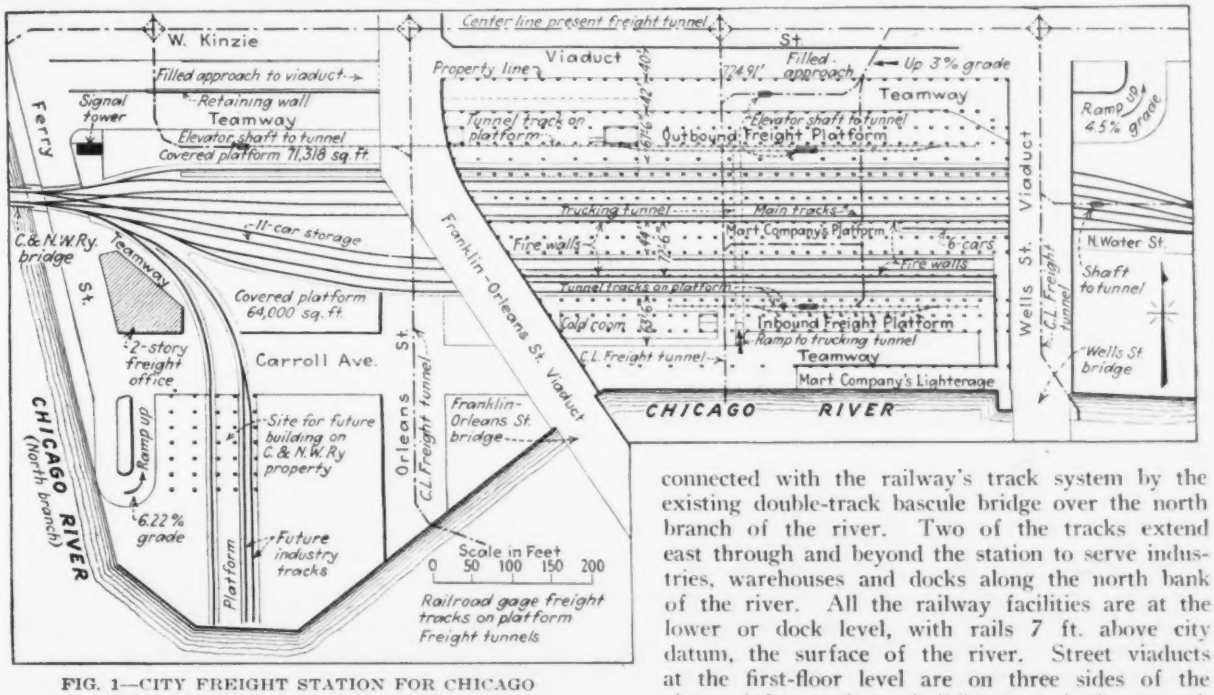


FIG. 1—CITY FREIGHT STATION FOR CHICAGO & NORTHWESTERN RAILWAY IN CHICAGO

A specially interesting feature of this terminal is that the air rights (from a level about 23 ft. above the water, or city datum) have been sold to Marshall Field & Company, a local drygoods concern, which organized a company to erect and operate an eighteen-story building for warehouse, office, salesroom and light manufacturing purposes. This structure, called the Merchandise Mart, is said to be the largest commercial building in the world, having more than 4,000,000 sq.ft. of floor space. To serve this warehouse, the railway has granted to the Mart company a part of the river-front dock or wharf for lighterage, and also the use of individual tracks in the freight station, with a platform on which elevators will be located to serve the floors above. A layout of the station is shown in Fig. 1, with a cross-section in Fig. 2.

One of the deep-level tunnels of the narrow-gage freight railway system of the Chicago Tunnel Company extends across the site, and branches will be built from this to elevator shafts in which the small tunnel cars of 24-in. gage can be handled directly to and from the station platforms and the upper floors of the building. There will also be a transverse trucking tunnel, 12x8 ft. in section, just below the track floor, reached by ramps of 15 per cent grade from the dock or wharf and 10 per cent grade from the railway company's basements under both the inbound and outbound freight platforms.

At the west end, the track layout of the station is connected with the railway's track system by the existing double-track bascule bridge over the north branch of the river. Two of the tracks extend east through and beyond the station to serve industries, warehouses and docks along the north bank of the river. All the railway facilities are at the lower or dock level, with rails 7 ft. above city datum, the surface of the river. Street viaducts at the first-floor level are on three sides of the site, and the warehouse building is set back to permit of a broad upper-level street along the riverfront, over the railway's wharf and tracks.

From the plan it will be seen that on the north side of the freight station, between it and Kinzie St., is a 40-ft. driveway along the outbound freight platform. At its west end the driveway opens into Ferry St. At its east end it rises on a 4.5 per cent grade to Kinzie St. by a ramp turning an angle of 90 deg.

The outbound freight platform, 65 ft. wide, is 1,350 ft. long, giving tailboard space for 128 trucks or wagons. Along it run two narrow-gage tracks, on each of which is a shaft to the freight tunnel elevators. Space is provided also for ordinary freight elevators up to the warehouse floors. Three outbound tracks are provided, each long enough for 31 cars and all extending east of the building to connect with the through tracks. The first and second tracks are spaced 16 ft. 6 in. c. to c. and between them is a 6-ft. trucking platform; the second and third tracks are 13 ft. c. to c. In the next bay are the two main or running tracks which extend eastward beyond the station, as noted above.

Adjacent to these through tracks is the space owned by the Mart company for its warehouse service, including a 45-ft. platform and two stub tracks for fifteen cars each. This space is inclosed by fire walls so as to be entirely separated from the railway company's freight facilities, a private trucking gangway being provided to connect with the Mart's space on the river-front wharf.

Next to the inclosed space the railway has two inbound freight-house tracks, both of which are stub tracks with capacity for 25 cars each. Adjacent to them is the inbound freight platform, 45 ft. wide, with a 40-ft. driveway on the outer or river side. The platform is 1,000 ft. long, with tailboard space for 96 vehicles. It has an elevator and two narrow-gage tracks for the tunnel cars. Beyond the driveway is the wharf. Beneath the two railway platforms are basements for storage purposes, and on the inbound platform is a cold-storage room with basement. No basement is provided under the warehouse company's platform at present.

The Merchandise Mart or warehouse building is 324 ft. wide, with a frontage of 724 ft. on Kinzie St. and 580 ft. on the river side. On this latter side space is provided for a broad high-level street along the river-front, this street being carried as a viaduct over the wharf and extending between the viaduct approaches of the Wells St. and Franklin-Orleans St. bridges. The building, with its first floor 23 ft. above water level, will be eighteen stories high, with a tower portion having five additional stories.

Auxiliary to the freight station will be a team yard with tracks at right angles to those of the station, and serving a proposed barge terminal. It will be at the dock level, approached from the viaduct by a U-shaped ramp having a grade of 6.22 per cent. Over this yard there may be erected a building as an air-right development and suitable for mercantile purposes. It may be served by two stub tracks on the outside, as indicated on the plan. At the junction of Carroll and Ferry Sts. there will be a two-story building of irregular shape for the offices of the railway company's freight terminal.

This large city terminal was designed under the direction of W. J. Towne, chief engineer, Chicago & Northwestern Railway, and the planning and construction are in direct charge of J. S. Pole, engineer of track elevation. The warehouse building was designed by Graham, Anderson, Probst & White, architects. The general contractor for the station and warehouse is the John Griffiths & Son Company. Tracklaying will be done by railway forces. It is expected that the freight terminal will be ready by July 1, 1930, and the warehouse building by May 1, 1930. The total cost is estimated at $2,000,000 for the railway facilities and $30,000,000 for the warehouse building as completed and equipped for use.

New Steel Floor Developed for Building Use

Steel Deck of Welded I-Beams and Plates Assures Large Reduction in Dead Loads—Affords Ample Strength at Reasonable Cost

By Lee H. Miller

Chief Engineer, American Institute of Steel Construction, Cleveland, Ohio

INVESTIGATIONS are under way on a new type of floor construction made up of standard structural beams and steel plates which it is estimated will improve the strength and stability of buildings, reduce the loads on columns and be available at a reasonable cost on all projects. An important factor making possible the development of the floor at the present time is the recent rapid advance of the structural welding art.

When the skyscraper came into existence the steel industry for some unaccountable reason furnished only

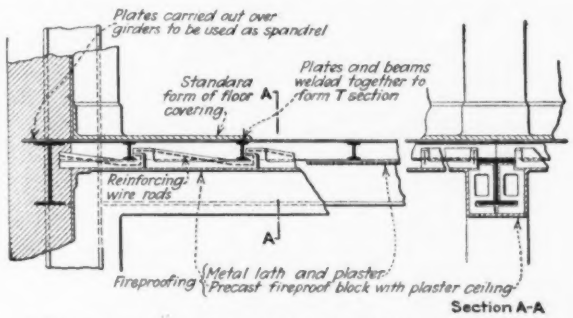

Plates carried out over girders to be used as spandrel
Standard form of floor covering
Plates and beams welded together to form T section
A
Reinforcing wire rods
A
Fireproofing
Metal lath and plaster
Precast fireproof block with plaster ceiling
Section A-A

FIG. 1—STEEL DECK FLOOR DESIGN WITH BEAMS AND PLATES CONTINUOUS
Note that beams are placed on top of girders.

the columns, girders and beams, leaving to the general contractor, the architect and the engineer the development of some means of filling in the floor panels. Originally this was done by brick arches, later by flat tile arches, and still later by other types of material such as wood, packed floors, reinforced concrete and gypsum. The present methods of floor construction involve the use of materials the dead load of which is considerably more than the live load which is to be supported and which consequently constitute one of the limiting conditions to the use of increased heights of multiple-story buildings. Reference to the column schedule of any tall building will reveal that the column stresses for dead loads are often approximately two-thirds of the total stress.

By the use of steel plates and ordinary structural steel beams, it is proposed to develop a welded steel floor of

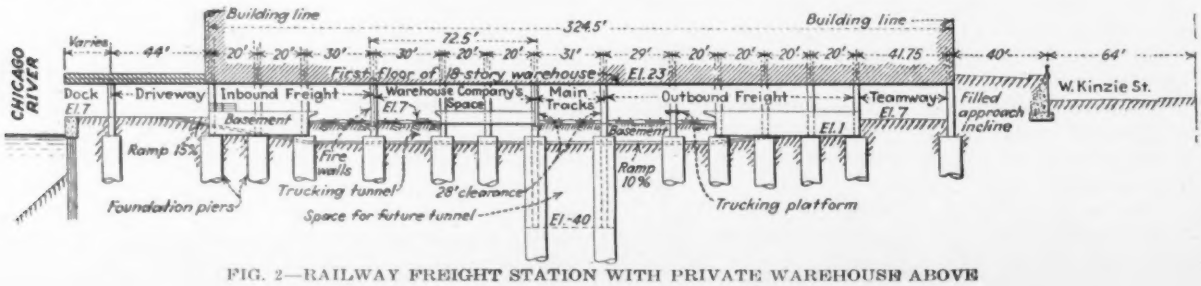


FIG. 2—RAILWAY FREIGHT STATION WITH PRIVATE WAREHOUSE ABOVE

light weight, in which it is possible to determine all stresses in advance. It is proposed to use 3-, 4-, 5-in. or other rolled I-beams of whatever depth is found necessary, spaced 24 in. apart and spanning from girder to girder. Along the tops of these beams will be laid continuous steel plates $\frac{3}{16}$ or $\frac{1}{4}$ in. thick and wide enough to leave a space between their edges approximately $\frac{1}{4}$ in. wide. The edges of the plates will lie along the center line of the webs of the small beams, and an automatic welding machine will work along this joint, welding the edges together and to the top flange of the beam.

The result will be a built-up T-section with the plate acting as the upper flange and the beam acting as the vertical leg. The neutral axis of the section will be close to the top flange of the beam, and when the lower flange of the beam is stressed to 18,000 lb. per sq.in., the plates and the top flange will have a stress of only 3,000 to 4,000 lb. per sq.in.

It is, of course, assumed that this combination construction will be made continuous over the tops of the supporting girders, or else that the small beams will be framed into the girders flush on the top with the floor plates continuous. Examples of two weights of floors with fixed end supports are given below:

1. By using 3-in. 5.7-lb. I-beams and a $\frac{3}{16}$-in. plate, the floor will carry a total load of 190 lb. per square foot

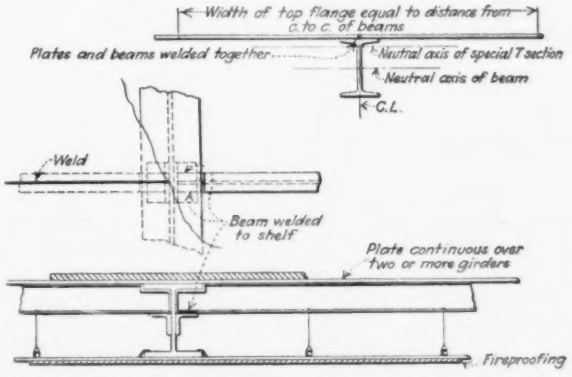

Width of top flange equal to distance from c. to c. of beams
Plates and beams welded together
Neutral axis of special T section
Neutral axis of beam
C.L.
Weld
Beam welded to shelf
Plate continuous over two or more girders
Fireproofing

FIG. 2—STEEL FLOOR CONSTRUCTION WITH PLATES CONTINUOUS
Note that beams frame into girders with top flanges flush.

on a 10-ft. span with a deflection of 0.07 in. The same floor will carry a total load of 85 lb. per square foot on a 15-ft. span with a deflection of 0.157 in. The weight of the steelwork, including beams and plates, will be $10\frac{1}{2}$ lb. per square foot.

2. By using 5-in. 10-lb. I-beams spaced 2 ft. centers and a $\frac{1}{4}$-in. plate, a total load of 240 lb. per square foot can be carried on a 15-ft. span with 0.098-in. deflection; on a 20-ft. span a total load of 135 lb. per square foot can be carried with a 0.174-in. deflection. The weight of the steelwork in this floor will be 15.2 lb. per square foot.

The strength and deflection of the plates themselves are quite satisfactory. A $\frac{1}{4}$-in. plate will carry a uniformly distributed load of 562 lb. per square foot, with a deflection of 0.115 in. between the small beams spaced 24 in. apart. Such a plate will also carry a concentrated load between the small beams of 750 lb. per linear foot with a deflection of 0.057 in. Under the same conditions a $\frac{3}{16}$-in. plate will carry a uniform load of 316 lb. per square foot, or a concentrated center load of 422 lb. per linear foot. These loads are in excess of what the composite T-section will develop and indicate that the floor construction is amply safe.

Wind Resistance—With this type of floor construction the solid steel deck will act as a girder to prevent any torsional distortion of the building when subjected to wind or earthquake action. The engineer can be assured that the deck flooring will deliver the stresses to the most rigid part of the vertical frame. Another advantage of this floor construction is that it can be carried beyond the columns to provide spandrel construction to support the outside walls.

Fireproofing—It will be necessary to fireproof the floor when the combustible contents of the structure will generate a heat that will produce a temperature in the steel in excess of 1,000 deg. F. This fire protection can be secured either by cast blocks of fireproofing inserted on the lower flange of the small beams or by metal lath and plaster ceiling. Either type of construction will provide openings for conduits which can be carried along the main girders and transversely under the steel plates between the small beams. On the other hand, if it is found advisable to do so, it will be possible to have a light fireproofing applied to the top of the steel plates, incorporating the various conduits which lay on top.

The cost of welding the bead between the edges of the plate and the top of the beams has been estimated by one contractor at less than 2c. per linear foot with the use of automatic machinery. Another contractor has stated that he is ready to do this work for between 3c. and 4c. per square foot of floor. Since the welding will be done a short distance from the neutral axis, the difficulty of the floor curling up due to shrinkage will be practically negligible. The total cost of a floor constructed of 3-in. I-beams, $\frac{3}{16}$-in. plates covered on top with cork tile and fireproofed on the under side with metal lath and plaster will be about $1 per square foot.

The only precedent for this steel T-section floor construction exists on the decks of battleships and on the charging floors of open-hearth furnaces. On both of these the plates, which are $\frac{3}{16}$ and $\frac{1}{4}$ in. thick, are subjected to the most severe service that any floor can receive. The proposed type of floor construction will weigh from 20 to 60 lb. less than any of the existing types now in use, and in connection with a proposed 75-story building with floor panels $21\frac{1}{2}$x$22\frac{1}{2}$ the saving in dead load on the foundations for each column will be nearly 2,000,000 lb.

For bridge floor construction also, a combination of steel beams with checker plates could be assembled that would very materially reduce the dead load of the structure and provide a lateral rigidity more satisfactory than any lateral floor diagonal now used. It will be found that checker plate floors are used at present to take care of the wear on wood floor bridges, and the question of their being slippery has never been raised. On the other hand, checker plate floors are used constantly for stair treads and landings to reduce slipperiness.

Russian Government Planning 3,040 Miles of New Railways

Railway extensions totalling 3,040 miles are planned by the Russian government within the next two years. Work has already been started on sections totaling 1,180 miles. A line connecting the Siberian Railway with the Persian frontier will be opened to general traffic in October. It is stated that in a distance of 1,050 miles only two major bridges were necessary and that during the month of June an average of $1\frac{1}{2}$ miles of track was laid daily.

THREE-YEAR-OLD CADDO PARISH ROAD WHICH CARRIES HEAVY TRAFFIC, INCLUDING LOGGING TRUCKS
Winnifield marble, 9-in. macadam, 18 ft. wide, cost $12,690 per mile. Oiled in 1926 with 250 penetration oil, 0.4 gal. per square yard, covered with sand and Joplin chats at a cost of 9c. per yard. Surface maintenance cost per mile: 1926, $170; 1927, $153; 1928, $18.50, and 1929 to April, $30.

Engineering in Caddo Parish, Louisiana

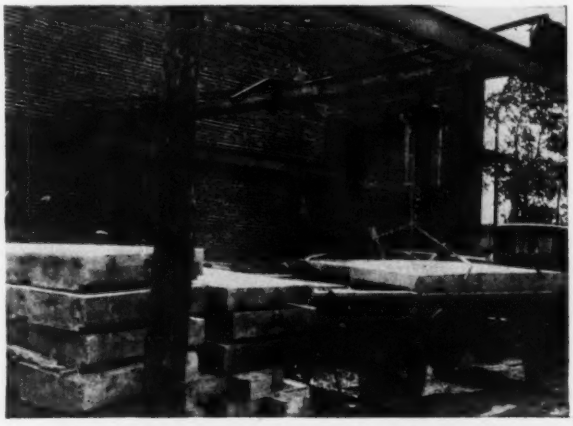
Engineer's Authority Limited to Highway Work—Over 400 Miles Improved Since 1912—All Is Stage Construction by Current Funds—Cost and Engineering Records Emphasized

WITH INCREASING HIGHWAY programs county engineering has risen in responsibility and in its demand for technical ability and organizing skill. Many counties have today engineering organizations rare a generation ago anywhere except in the railway field. To exemplify this development of engineering service articles will be published from time to time describing modern county engineering organization and planning. The initial article, on Caddo Parish in Louisiana, tells of a county starting from zero in improved road mileage and now having 400 miles and an annual highway improvement budget of more than $800,000. Its record of financing, planning and construction is only that possible to any ambitious county of moderate wealth. —EDITOR.

IN LOUISIANA the political subdivisions which correspond to the counties of other states are called parishes. Caddo Parish, in which Shreveport, the second city of the state, is located, rather pridefully presents the claim for distinction that all its public works are financed from current funds. Not only does it not borrow to build and maintain its highways, which are its chief public works, but it follows a pay-as-you-go policy in its drainage and levee work and in its public building construction. No bonded indebtedness has been incurred for anything, a $1,250,000 court house recently completed being paid for in three annual installments, the money coming in about as fast as construction proceeded. A levee system 90 miles long and 125 miles of drainage ditches with one canal 140 ft. wide at the bottom have been paid for as built. The engineering on levees and drains is handled by the state engineer's office through local district boards. The parish engineer has charge of the road work only.

With all credit to the parish financing of public works, these works deserve more particular notice for their progressive engineering. A consistent plan of stage construction of highways is being carried out. An even and logical balance is preserved between contract and force-account road improvement. A complete engineering cost-accounting system is followed and orderly records are at hand of all engineering activities. Finally, a technique of oil-treating traffic-bound and rolled stone and gravel roads has been developed which has received wide publicity. Incidental to road improvement there has recently been developed a precast concrete slab and pile bridge design of notable interest.

Parish Geography and Industry—Located in the northeast corner of the state, Caddo Parish lies between Texas on the west, Arkansas on the north and the meandering

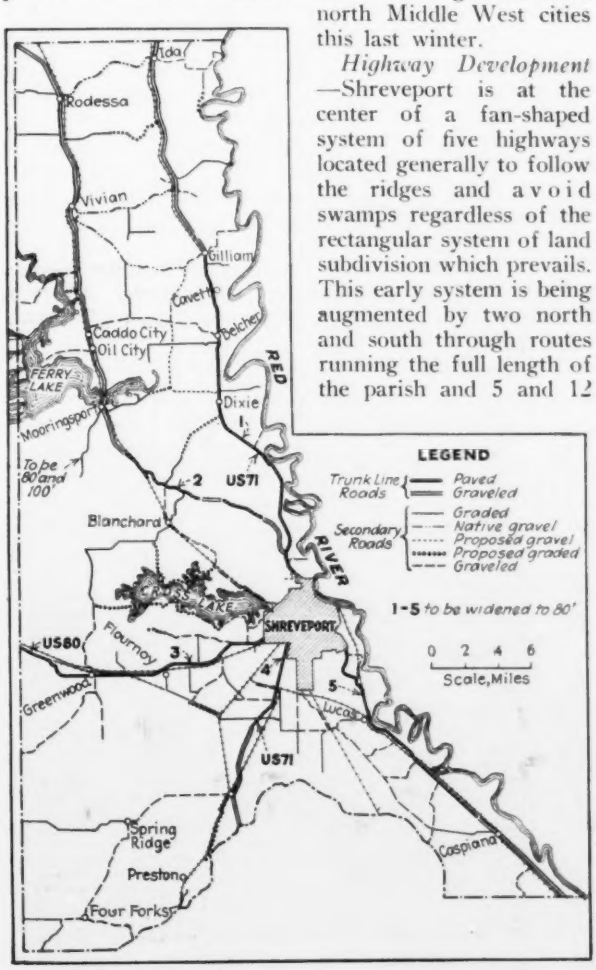


LOADING 3x10-FT. PRECAST BRIDGE SLABS

Red River on the east. It is long and narrow, 56 miles long north and south and from 13 miles wide at the north end to approximately 25 miles at the south end. Shreveport, the principal city of the parish and of this part of the state and claiming a trade territory 100 to 200 miles in extent, is located on the river 17 miles from the southern border of the parish. The river valley is 1 to 5 miles wide, of rich alluvial bottom lands 200 ft. above the Gulf level and protected by river levees from overflow. The adjacent hills rise 50 to 100 ft. higher. The location and requirements, present and potential, of Shreveport have much to do with the transportation problems of the parish. The city, which is the seat of the parish government, contains 80 per cent of the 100,000 people of the parish. The population has nearly doubled since the last census. Several radiating railroads tap here the center of the wealth of natural resources.

Briefly, the Red River valley produces the greatest cotton crop and finest pecans in Louisiana. Natural gas and oil pools have brought many refineries and a $3,000,000 glass factory. A 16-in. gas line leads north to Little Rock. The parish has some reputation as a health resort, four sanitariums being located there. Though the tourist trade is nominal, it is growing because of the unexcelled fishing and hunting in the numerous lakes and forested areas. East and west through traffic over U. S. highway 80 has developed rapidly, this having been one of the preferred routes to California from Chicago and other north Middle West cities this last winter.

Highway Development—Shreveport is at the center of a fan-shaped system of five highways located generally to follow the ridges and avoid swamps regardless of the rectangular system of land subdivision which prevails. This early system is being augmented by two north and south through routes running the full length of the parish and 5 and 12 miles west of Shreveport, by a belt line running generally around the city, by a scenic route around Cross Lake and by numerous cross-roads 3 to 5 miles apart serving local needs.



CADDO PARISH, LOUISIANA, HIGHWAY SYSTEM

STORAGE YARD UNDER CRANE FOR BRIDGE UNITS

It is the development of this system from a single mile of gravel road in 1912 carrying no automobiles to 318 miles of improved all-weather roads and 116 miles of well-graded and drained roads that has claimed the greatest attention of the parish engineer. In this mileage there are 21 miles of concrete, 47 miles of bituminous pavements, 44 miles of oil-surfaced macadam and 135 miles of gravel, nearly all of which is oil surfaced.

This system, costing approximately $10,000,000, has been built in stages, utilizing low-cost road type to get mileage which added development and permitted higher types to come in later to meet the increase in traffic. For all except a small mileage of local gravel roads in the northern end of the parish, all road metal, concrete materials and asphaltic oils have had to be shipped in.

The work has been financed by a five-mill tax which brought in $125,000 the first year; the income now amounts to $800,000 annually, but includes "severance" or royalty taxes on oil and gas and certain other minor allocations from federal and state aid. The five-mill tax continues to be voted at the expiration of each five-year period for which it runs.

Stage Construction—With a small amount of money available at any one time, stage construction was about the only policy that could be followed.

Work on the nearest 5 miles of the five principal roads leading out of Shreveport was undertaken first. Then the next 5-mile zone was improved, and so on. Generally the roads were graded 24 ft. wide. For a width of 16 ft. the surface was covered with gravel or macadam. For the first five years the cost, including grading, concrete culverts and wooden bridges, was $6,500 per mile.

A sand-clay gravel was used containing 50 to 60 per cent of rounded pebbles retained on a 10-mesh screen up to 2 in. in diameter and 40 to 50 per cent of red clay and sand. After six months' to a year's service oil was applied to the road after it was patched, bladed to a smooth surface and swept clean. From ⅓ to ½ gal. per square yard was applied and covered with the material previously swept from the road. The macadam was of soft limestone or marble; it was oiled with better results than the gravel. When traffic increased, 6 in. of macadam was put on top of the gravel, and this in turn was covered with a 2-in. surface of asphaltic concrete,

BRIDGE WITH PRECAST PILE BENTS AND STEEL STRINGERS CARRYING PRECAST FLOOR SLABS

amiesite, sheet asphalt or other type, using the old macadam as a base. Widening to 18 and 20 ft. took place with the change to higher types.

Recently the engineer has been using a soft limestone from Arkansas which works like caliche, blading quite smooth and taking a set after a time. Oil on this surface, which would dust badly under traffic, makes a smoother riding road than any of the other materials. This material is being used this year for a large mileage of one of the main roads.

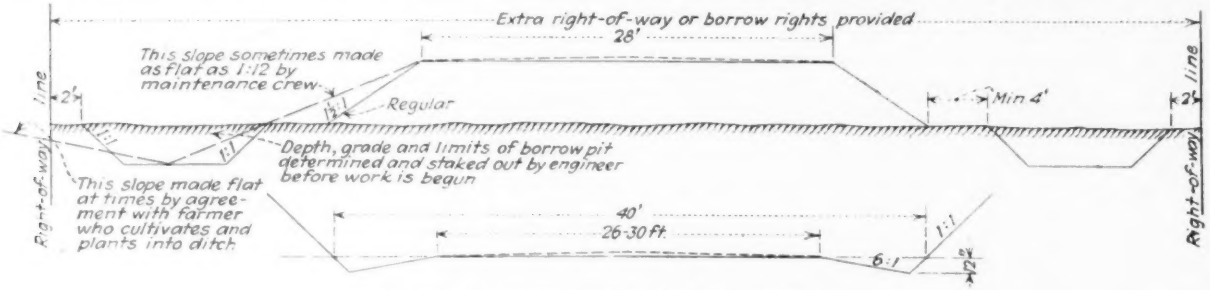
Bridge Practice—The prevailing treacherous foundations have restricted bridge types to slabs on pile bents. End abutments are not used, because of the ease with which much of the material flows, the bents being carried far back into the solid materials of the approach roadways.

Much use is made of creosoted timber bridges with all cut surfaces thoroughly swabbed and then protected with galvanized-iron covers or heavy tar paper. Dating nails are driven into the various members for easy reference.

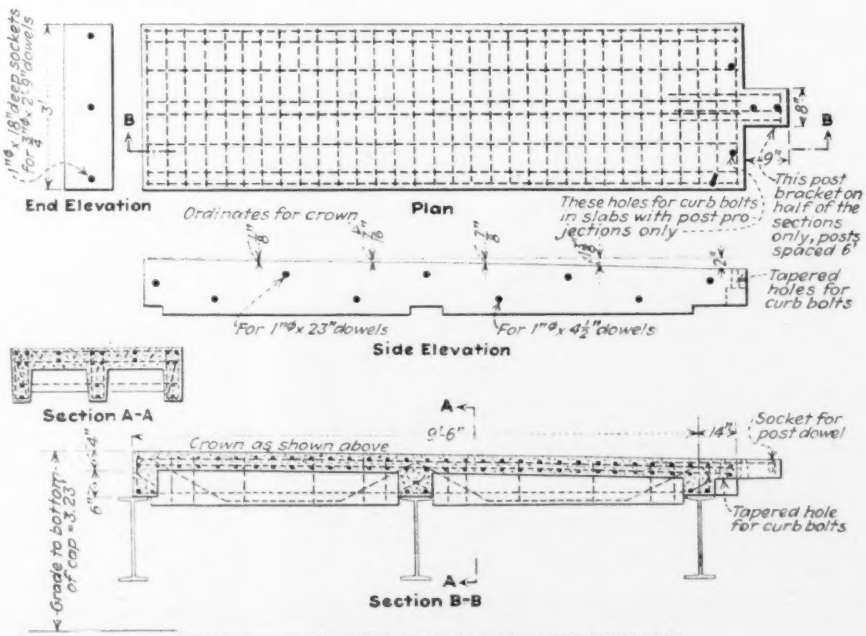
Recently a pile-bent, precast concrete bridge has been developed. Concrete piles are driven 10 to 15 ft. deep and the heads are then broken down to expose an anchor bolt which fits into a hole in the cap through which concrete is poured to re-form the new top at the proper elevation. Over the concrete caps are laid 18-in. steel I-beam stringers covered with no-oxide paint, a crude vaseline. Concrete floor slabs 4 in. thick, 3 ft. wide and 10 ft. long rest on the stringers and are doweled together. These slabs have flanges 10 in. thick on all four sides, the center being cored out like pan ceilings in buildings. Joints between adjacent slabs are filled with sulphur, the bottom being closed with stiff clay and the ends with oakum. Thus the bridge is ready for use as soon as completed. To make the bridge conspicuous to travel, the timber rail boards are painted white and the concrete posts and wheel guards are painted black, the latter with cut-back asphalt.

The concrete and creosoted timber bridges cost about the same. The concrete units are cast in the parish storage yard at a cost of $20 per cubic yard or less. All culverts are concrete pipe; they also are made in the storage yard under cover by a gang working continuously.

Storage Yard—The parish maintains a well-equipped yard in Shreveport where the precast concrete units are stored under a crane runway for quick loading. Here also are oil tanks, gasoline storage, machine and carpenter shops and space for garaging the 35 trucks and automobiles owned by the parish and such other miscellaneous equipment as oilers, graders and camp wagons. A supply of small tools, spare parts and groceries for the camps is also kept on hand.


Extra right-of-way or borrow rights provided
28'
This slope sometimes made as flat as 1:12 by maintenance crew
Regular
Min 4'
2'
Right-of-way line
Depth, grade and limits of borrow pit determined and staked out by engineer before work is begun
This slope made flat at times by agreement with farmer who cultivates and plants into ditch
40'
26-30 Ft.
1:1
6:1

STANDARD GRADING SECTIONS ON HIGHWAYS


End Elevation
Plan
Side Elevation
Section A-A
Section B-B
This post bracket on half of the sections only, posts spaced 6'
These holes for curb bolts in slabs with post projections only
Tapered holes for curb bolts
Socket for post dowel
Crown as shown above
Grade to bottom of cap = 3.23'
For 1"ø x 23" dowels
For 1"ø x 4½" dowels

PRECAST CONCRETE DECK SLABS FOR BRIDGES

DISCARDED OIL PIPE USED FOR RIGHT-OF-WAY MARKERS

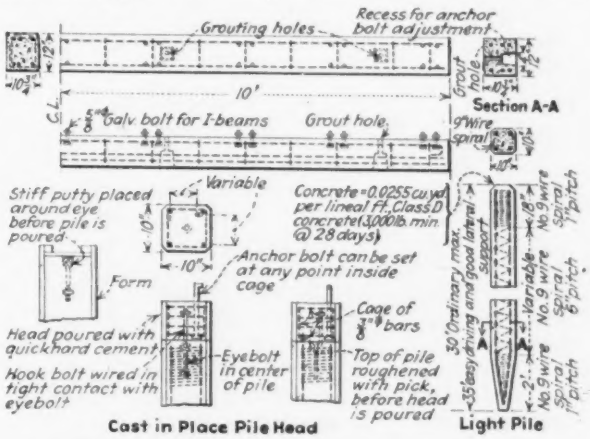
The storage yard foreman maintains a system of quick-reference, card-index inventorying for all material and equipment on hand and in the various camps; the supply is checked up frequently. Nothing is issued except on order and every tool must be accounted for. A complete inventory is made once a year. Equipment, such as oilers, graders and excavators, is charged on a rental basis to the definite piece of road under construction or maintenance, everything ultimately being figured back to the unit basis for ready comparison.

In the main office these records are worked up into monthly sheets which are checked with expenditures by the auditor of the parish. Early in the year a budget for each road is decided upon by the police jury and it is the engineers' duty to watch the expenditures and keep both unit costs and totals within the limits set. Every piece of equipment and even every mule has a history card on which is entered original cost and subsequent items of maintenance. A lazy mule stands as little chance of staying on the job as a poor workman.

Maintenance—All maintenance is by force account, as is grading, graveling, building macadam roads and oiling. The higher types of roads, such as concrete and asphalt pavement, are built by contract. The maintenance crew organization must be kept up in winter, when heavy rains make almost continuous attention necessary. It is believed, therefore, that for the lower-type roads which are unsatisfactorily handled, at least by contractors of this section, the day-labor basis with trained crews is the best plan. The engineer claims that he saves the contractor's profit, but this is done only by eternal vigilance, an elaborate cost-accounting system and a well-developed esprit de corps through competition between outfits. On a resurfacing job comparative costs per cubic yard are kept of the following items: subgrade, unloading, hauling, spreading, rolling, sprinkling and time of foreman. A monthly bulletin of half a dozen pages giving camp gossip, records and achievements helps to unify the organization and keep up the morale.

Miscellaneous Practices—Sleeping wagons for the camp outfits have been built in the shops. In an oil-well country large quantities of oil-drill pipe are available. The 2- and 3-in. sizes are cut into 6-ft. lengths, painted white and used as right-of-way markers. These pipes tend to ward off encroachments and the stenciled notations as to location give inspectors an easy means of reference in their reports.

A right-of-way map is made up on which are noted all promises (trees to be saved, entrances, etc.) made to property owners who have dedicated land for new roads or for widening old ones. Widening to 80 and 100 ft. is now taking place on many of the main roads. This is done less for immediate traffic needs and wider pavements than for easier and better maintenance of ditch shoulders. Slopes up to 1 on 12 are used. One observant patrolman has discovered, quite by accident, that dragging a discarded tangle of fence wire in a ditch cleans and smooths it better than any of the regular tools. The bundle of wire acts like a housewife's metal-ribbon pot cleaner.


Grouting holes
Recess for anchor bolt adjustment
Grout hole
10'
Section A-A
Galv. bolt for I-beams
Grout hole
9 Wire spiral
Stiff putty placed around eye before pile is poured
Variable
Concrete = 0.0255 cu yd per lineal ft. Class D concrete (3,000 lb. min @ 28 days)
Anchor bolt can be set at any point inside cage
Form
Head poured with quickhard cement
Hook bolt wired in tight contact with eyebolt
Eyebolt in center of pile
Cage of bars
Top of pile roughened with pick, before head is poured
30' Ordinary max.
35' easy driving and good lateral support
No. 9 wire spiral
Cast in Place Pile Head
Light Pile

PRECAST PILE AND CAP DETAILS FOR BRIDGES

Administration—To carry out and direct all of these duties the police jury of Caddo Parish employs J. T. Bullen, who came first to the parish in 1911, organized the work and stayed until 1916, when he went with the U. S. Bureau of Public Roads as district engineer. He returned nine years ago. He has four field parties under two assistants who check up on the parties every day, one in the north end, the other in the south end of the parish. Five large outfits or camps with fifteen to twenty teams are maintained, and several small ones. Two tractor outfits for rough work, several small tractors, two rollers, two oilers and 25 trucks are usually employed, all under the direction of a superintendent and field engineer, J. W. Barnwell.

The yard is handled by Mr. Perryman, a bookkeeper and foreman who acts as general superintendent of the

SLEEPING WAGONS OF WOOD AND GALVANIZED IRON FOR CAMP OUTFITS

yard office and precasting plant. In the courthouse is an office and bridge engineer, W. J. Fulton, who has two draftsmen. A bookkeeper, a cost accountant and a stenographer complete the office personnel. Finally, mention must be made of the handsome metal furniture, file and map cases which make for the best working conditions for the staff and enable the engineer to keep his records at his fingertips at all times. As a result of this organization and its wholehearted backing by the officials, Caddo Parish can proudly boast of a widespread system of excellently maintained roads, efficiently managed at comparatively low cost.

Six Presidential Approval Orders on Mississippi Flood Control

President Coolidge Did Not Authorize Payments for Lands or Flowage Rights in Boeuf and Atchafalaya Floodways

RECENT inquiry into the nature and terms of the Presidential approval of the Mississippi flood-control project as provided for in the Jones-Reid flood-control act of May 15, 1928, brought to light the existence of six separate executive orders. These orders are not mere approvals, but lay down the basis of land payments for levee rights-of-way and floodways. In certain cases they limit the amount of money that may be paid for floodway lands and in other instances provide that no money shall be paid for floodway lands or flowage rights.

According to Section 1 of the flood-control act, the Jadwin plan was adopted, but an engineering board was authorized to consider the differences between the Jadwin plan and the Mississippi River Commission plan, to make "such further surveys as may be necessary," and to recommend to the President action in respect to engineering differences between the two plans. It provided that the decision of the President upon all recommendations or questions submitted to him by such board shall be followed in carrying out the project. According to the recent opinion rendered by Attorney-General Mitchell (*Engineering News-Record*, Aug. 1, 1929, p. 186), the board could consider and the President could approve only matters of engineering difference between the two plans.

The several executive orders of approval, extending from Aug. 13, 1928, to Jan. 16, 1929, are as follows:

Order of Aug. 13, 1928

The policy and method of dealing with the problem set out in the report, dated Aug. 8, 1928, of the board provided for in Section 1 of "An act for the control of floods on the Mississippi River and its tributaries and for other purposes," approved May 15, 1928, hereto annexed, are approved. The balance of the report is approved excepting and reserving for my future action those parts which contemplate the acquiring of rights in land for constructing spillways and floodways.

Order of Nov. 21, 1928

Supplementing my approval on Aug. 13, 1928, of the report of the board provided for in Section 1 of "An act for the control of floods on the Mississippi River and its tributaries and for other purposes," approved May 15, 1928, which approval excepted and reserved for future action those parts of the report which contemplated the acquiring of rights in land for constructing spillways and floodways, the construction of the spillway provided for in the adopted project on the east side of the Mississippi River north of New Orleans is approved at the site opposite Mile 936 south of Cairo.

Land for rights-of-way for the spillway structure, land from the existing main river levee line to said structure and land for the limiting levees of the floodway into which the spillway will empty will be secured by condemnation as authorized by law. The purchase of flowage rights over the land within the floodway between the limiting levees is authorized, provided that in no case shall the purchase price be more than three times the present assessed valuation of the land.

Order of Dec. 11, 1928

Supplementing my approval of Aug. 13, 1928 . . . the construction of the back (westward) levee of the Birds Point-New Madrid floodway provided for in the adopted project on the west side of the Mississippi River opposite Cairo, Ill., is approved.

Land for rights-of-way for this levee will be secured by condemnation as authorized by law. The purchase of flowage rights over the land within the floodway between the existing riverside levee and the back (westward) levee is authorized, provided that in no case shall the purchase price for the flowage on the land above the backwater area in the southern part of the floodway be more than 66 per cent of the present assessed valuation of this land and provided that in no cases shall the purchase price for the flowage of this land within the backwater area be more than a price ranging from 66 per cent of the present assessed valuation of the land at the upper limit to zero at the lower limit of the backwater area within the floodway.

The purchase of the land within the floodway immediately adjacent to the proposed relief section of the riverside levee (about 3,200 acres) is authorized, provided that in no case shall the purchase price be more than twice the present assessed valuation of the land.

The construction of the relief section proposed for the riverside levee is authorized, provided that it shall not be made until flowage rights have been secured on at least 50 per cent of the area above backwater and 50 per cent of the area below backwater within the floodway.

Order of Dec. 13, 1928

Supplementing my approval of Aug. 13, 1928 . . . the construction of the levee from near Bordelonville, La., to near Hamberg, La., on the Bayou des Glaizes, in accordance with the adopted flood-control project is approved.

Lands for rights-of-way for this levee will be secured by purchase if reasonable prices can be secured; otherwise this land will be secured by condemnation. The purchase of either the land or the flowage over same east of the new levee proposed and within the present levee line on the Bayou des Glaizes is authorized, provided the price paid in any case shall not be over 2½ times the assessed value of the land at the present time. The breaching of the levee line around this land in accordance with the adopted project will not be done until 50 per cent of the land or flowage has been secured.

Order of Jan. 10, 1929

Supplementing my approval of Aug. 13, 1928 . . . the construction of the protection levees in the Boeuf basin, as provided for in the adopted project, is approved.

Land for rights-of-way for these levees will be secured by condemnation as authorized by law, provided that these lands may be purchased, when they can be thus secured at reasonable prices, which shall not, in any case, exceed 2½ times the assessed valuation of the present time.

Order of Jan. 16, 1929

Supplementing my approval of Aug. 13, 1928 . . . the construction of the protection levees in the Atchafalaya basin, as provided for in the adopted project, is approved.

Land for rights-of-way for these levees will be secured by condemnation as authorized by law, provided that these lands may be purchased, when they can be thus secured at reasonable prices, which shall not in any case exceed 2½ times the assessed valuation at the present time.

It appears from these several orders that the President did not authorize payment for floodway lands (or flowage rights) in the Boeuf and Atchafalaya floodways.

Roumania Planning a Ship Canal

Roumania is planning the construction of a ship canal connecting Bucharest with the Danube River. The project will include a hydro-electric plant, power from which will be used on the railway between Bucharest and Kronstadt, which is to be electrified. The entire project will cost approximately $31,000,000, payment of which is guaranteed by the government. As the size of the project places it beyond the powers of Roumanian contractors, international bids will be invited.

Additions to Mechanical Equipment at Baltimore Sewage-Pumping Station

Present Facilities at Eastern Ave. Plant, Consisting of Three 27,500,000-Gal.-Daily Triple-Expansion Pumping Engines, Have Been Supplemented by a 30,000,000-Gal.-Daily Turbo-Driven Centrifugal Pump

By C. E. Keefer
Engineer of Sewage Disposal, Bureau of Sewers, Baltimore, Md.

SUCH is the topography of Baltimore that before the sewage of the city reaches the sewage-works approximately half the flow is pumped at the Eastern Ave. station. This station, situated on the outskirts of the business and industrial center of the city adjacent to the harbor, is provided with three vertical triple-expansion pumping engines, each with capacity of 27,500,000 gal. daily (27½ m.g.d.) against a total head of 72 ft. Within recent years the sewage flow, augmented by the occasional infiltration of stormwater into the sanitary sewers, increased spasmodically to such a degree that additional pumping facilities were necessary. A number of projects to accomplish this purpose were considered, and of these the plan adopted consisted of installing one 30-m.g.d. non-condensing turbo-driven centrifugal pump.

The sewage flows to the pumping station in two interceptors and passes first through cage screens and then through bar screens with ¾-in. clear openings. Immediately beyond the latter screens are the suction wells, from which the sewage is pumped into two 42-in. cast-iron force mains. These mains are about 5,500 ft. long and discharge into a 99-in. gravity interceptor, tributary to the main outfall sewer.

At normal capacity, the pumping engines have a speed of 20 r.p.m. They use saturated steam at 175-lb. pressure, and operate under an exhaust pressure of about 26 in. of vacuum. The steam is generated in four boilers, two of which are kept in continuous operation. Three of these units have a rated capacity of 266 hp., the fourth 400 hp. The more important auxiliary equipment consists of two circulating pumps, which deliver harbor water for condensing purposes; two electric generators, which supply the station with current; an economizer for raising the temperature of the boiler-feed water, and coal- and ash-conveying equipment.

Under normal conditions two pumps are required to handle the sewage from about 9 a.m. to 9 p.m. During the remainder of the day only one pump is in operation. The average sewage flow per day from 1914 to 1928 is given in Fig. 2, and the average hourly rate of flow during 1926 is shown by Fig. 3. Previous to 1926 two unusually high flows occurred, the first on Oct. 9, 1922, when a maximum rate of 57 m.g.d. was reached, and the second on Feb. 20, 1924, when 60 m.g.d. was pumped. Fortunately, only two of the three pumps were required on these two occasions. A third high flow, however, occurred on Nov. 16, 1926, when a peak of 84 m.g.d. was reached. At this time all three pumps had to be put into service and were operated under an overload of 30 per cent for two hours. If one of the pumps had been undergoing repairs at the time, the results would have been disastrous. The sewage would have continued to rise in the sewers leading to the pumping station and would have flooded the basements and cellars of hundreds of business and manufacturing establishments, with a large property loss. Each of the above-mentioned high sewage flows was due to excessive quantities of stormwater getting into the sanitary sewers. This augmented flow was not confined to any one part of the city. It is believed that much of the stormwater entered the sewers through pipe joints and holes in manhole covers.

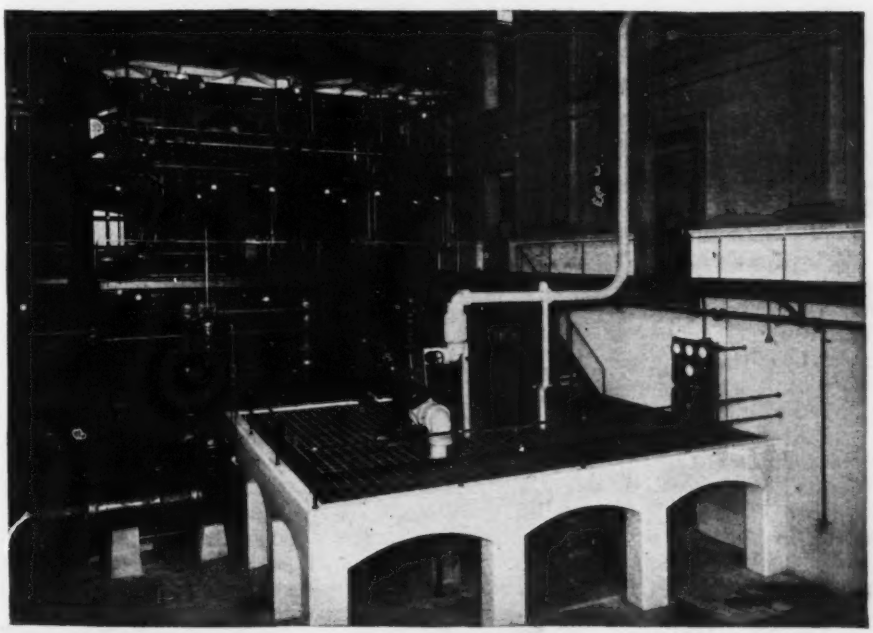


FIG. 1—NEW TURBO-DRIVEN 30-M.G.D. CENTRIFUGAL PUMP, WITH OLD TRIPLE-EXPANSION PUMP IN BACKGROUND

After the peak flow of 1926 additional pumping facilities became imperative. A study was therefore made to determine the most desirable type of equipment. There were many different ways of solv-

ing the problem, of which four seemed most feasible. Cost studies were therefore prepared to determine which of these was the cheapest. The type, number and capacities of the proposed pumping units included in each of these four projects are given in the accompanying table. Project 1 consisted of using the three existing pumps and installing one turbo-driven centrifugal pump with a capacity of 30 m.g.d. when operating at a total head of 70 ft. Since the triple-expansion pumps could be operated more cheaply than the proposed pumping unit and since they are of ample capacity to handle the normal sewage flow for many years, a non-condensing turbine was considered the most desirable to install. A turbo-driven pumping unit of the condensing type with all auxiliaries would cost nearly three times more than a unit of the non-condensing type. Purchase of the former unit, therefore, would not have been desirable. The cost of operating either of these units would be about the same, as it would be run only a few times a year to pump high sewage flows and at intervals to see that the equipment was in good condition.

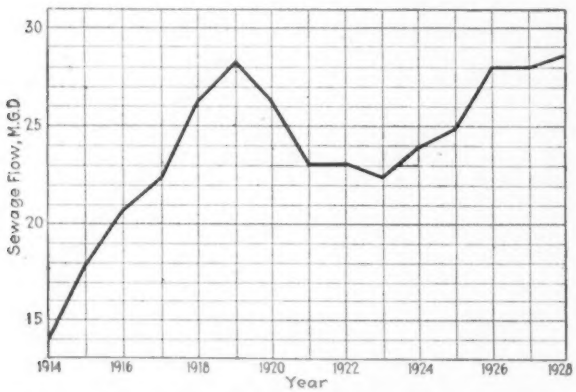


FIG. 2—SEWAGE FLOW AT EASTERN AVE. SEWAGE-PUMPING STATION, BALTIMORE

Project 2 is the same as project 1 except that the proposed pump was to be motor driven instead of steam driven.

The plan as embraced under project 3 comprised abandoning the three existing pumping engines and replacing them with two 20-, one 25- and two 30-m.g.d. motor-driven centrifugal pumps. The plan consisted of providing the station with three separate sources of power to insure continuity of service. These triplicate power facilities were considered necessary, for if the pumping station were to shut down for as much as 30 minutes, there would be no way to bypass the sewage, as the invert of the interceptor, which discharges at that station, is 13 ft. below mean low tide. Two of the sources of power were to connect with duplicate banks of 2,300/13,000-volt transformers, and the third power line, which was to provide direct current, was to operate a motor-generator set in order to furnish 2,300-volt alternating current.

The difference between projects 3 and 4 was that in the latter the pumps were to be operated by multi-stage steam turbines of the condensing type. The project also included installing superheaters in the existing boilers, and new condenser and boiler-feed equipment.

It is apparent that project 1 is the cheapest from an operating standpoint and also in so far as first cost is concerned. Adoption of either project 3 or 4 would have meant abandonment and removal of the existing triple-expansion pumping units. These units cost $450,000 in 1911 when new, and they are now worth considerably more at present-day prices. On the whole, project 1 was considered the most desirable of the four schemes, and it was therefore adopted. The station is thus entirely independent of outside sources of power. With the additional pumping facilities it is believed that the present equipment will handle the sewage flow for many years.

The new pumping unit does not possess any unusual characteristics. The impeller and wearing rings are of bronze and are designed to pass a 4-in. sphere. It was not felt necessary to use a pump with larger passages, as the sewage must be screened before it flows to the triple-expansion pumps. The duty of the unit when tested in the factory was 29.90 lb. of steam per brake-horsepower-hour, obtained with an exhaust steam pressure of 3 lb. per sq.in. Fig. 1 shows the new pumping unit with one of the triple-expansion pumps in the background.

The pumping unit stands on a structural steel platform, the top of which is 9 ft. above the original floor of the building. The structural steel is incased in concrete, which is faced with white enamel brick. In order that the vibration in the foundation due to the running of the pump might be a minimum, the horizontal members of the steel work were designed to have a maximum deflection of not over 0.02 in. Furthermore, the maximum horizontal deflection at the top of the platform was not allowed to exceed 0.02 in. when a horizontal force equal to the dead load of the turbine set was applied along the center of the turbine shaft and also at right angles to the shaft axis. Sufficient space has been provided beneath the platform so that condensing equipment can be installed at a future date.

COMPARATIVE ESTIMATED COSTS OF PUMPING EQUIPMENT, EASTERN AVE. SEWAGE PUMPING STATION, BALTIMORE, MD.

	Project 1	Project 2	Project 3	Project 4
Number and capacities of new pumping units.	One 30 m.g.d.	One 30 m.g.d.	Two 20 m.g.d. One 25 m.g.d. Two 30 m.g.d.	Two 20 m.g.d. One 25 m.g.d. Two 30 m.g.d.
Type of new pumps.	Turbo-driven centrifugal	Motor-driven centrifugal	Motor-driven centrifugals	Turbo-driven centrifugals
Number, capacity and type of existing pumps to be retained.	Three 27-m.g.d. vertical triple-expansion pumping engines	Three 27-m.g.d. vertical triple-expansion pumping engines	.....	.....
Average total head	70 ft.	70 ft.	70 ft.	70 ft.
First Cost of New Equipment:				
Pumping units	$12,000	$9,200	$89,500	$202,000
Suction and discharge piping	11,500	11,500	76,500	76,500
Bypass	42,000	42,000	11,060	11,060
Steam piping	230	.....	.....	13,800
Transformers	.....	3,900	16,120	.....
Miscellaneous	11,700	11,700	17,000	18,000
Total investment	$77,430	$78,300	$210,180	$321,360
Operating Costs Per Year:				
Interest and depreciation	$4,710	$4,650	$13,680	$24,020
Labor	42,100	42,100	22,810	34,930
Coal	22,890	22,890	4,280	29,760
Electric current	.....	2,360	29,230	.....
Miscellaneous	6,900	6,900	2,500	3,890
Total operating cost	$76,600	$78,900	$82,500	$92,600

Bypass Facilities—Even though pumping equipment as outlined in any of the above-mentioned projects were installed, there would be an element of danger in operating the pumping station, a hazard that has existed since the plant was put in operation. When the station was built, no provision was made to pump the sewage during an emergency into the harbor through a bypass. In case one of the two force mains were to break, which happened in December, 1918, the operation of the station would be

in grave jeopardy, and in case both mains were to fail, the repairs could be made only at a great cost and inconvenience to the public. It would be necessary to stop the pumps sufficiently long to drain the force mains and install new pipes. During this time a great number of cellars would be flooded.

To prevent such a catastrophe a bypass was constructed from the pumping station to the harbor. This bypass consists of cast-iron and precast-concrete pipe. Connections together with the necessary valves are provided to each pumping unit in the station. The valves, which control the discharge of sewage into the harbor, will be sealed, and the state Board of Health notified in case they have to be opened.

Auxiliary Equipment—At the time the pumping station was constructed two engine-driven centrifugal pumps, each with a maximum capacity of 4½ m.g.d., were installed to circulate harbor water for cooling purposes through the condensers of the engines which drive

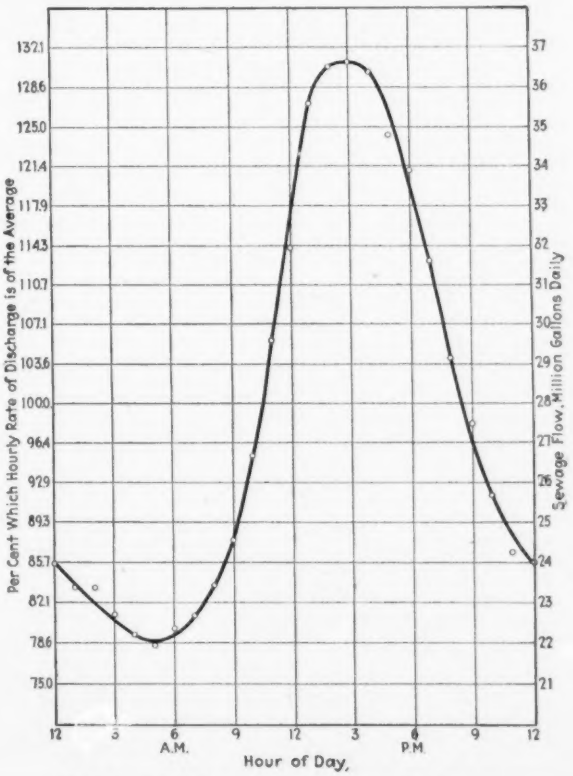


FIG. 3—HOURLY VARIATION IN FLOW OF SEWAGE FOR 1926

the main sewage pumps. There were also two engine-driven 35-kw. direct-current generators, which furnish all the electric power used in the pumping station. One of these power units was replaced in 1922 by a turbo-driven generator of the same capacity.

As it was believed that this old equipment had outlived its usefulness, tests were made under the direction of C. F. Goob, then mechanical-electrical engineer of the city, to determine the efficiencies of the three units. These tests indicated that new equipment should be installed. Two new units, consisting of centrifugal pumps direct connected by reduction gears to steam turbines, have been provided. The old engine-driven generator was replaced by a turbo-driven unit of standard design, made to operate at a back pressure of 3 lb. per sq.in. The exhaust steam is carried to an open feed-water heater for increasing the temperature of the boiler-feed water. Before these auxiliaries were installed, it was necessary to have three enginemen, one on each watch, to operate the old auxiliary equipment. Since then these men have been dispensed with at an annual saving of $4,800, as the new equipment requires little attention.

During the first five years the pumping station was in operation, the condensate from the main pumping engines was returned to the boilers. As the condensate contained oil in suspension and solution, a large number of boiler tubes blistered and had to be replaced. From 1917 until recently the condensate has been discharged into the harbor, and water from the city mains has been used in the boilers. This method of operation had two disadvantages, although the boiler-tube problem was solved. First, large quantities of water, costing $1,300 yearly, had to be continually used. By returning the condensate to the boilers it is estimated that this expenditure will be reduced to $270 yearly. Moreover, the condensate had an average temperature of 87 deg. F., 37 deg. higher than the temperature of the water from the city mains. Less coal will naturally be needed when using the condensate than when using city water.

Several means were considered to remove this oil continually. It was finally decided to install pressure-type filters, which consist of two steel drums 5½ ft. in diameter and 6½ ft. high. These filters contain gravel and graded sand. Before the condensate reaches the filters a coagulant consisting of alum and lime is added to emulsify the oil, which is retained on the surface of the sand. Periodically the filters are backwashed in order to clean them. After passing through the filters the condensate flows to the feed-water heater, and from there to boiler-feed pumps. The temperature of the water entering the boilers, which formerly was about 140 deg. F. before the feed-water heater was installed, now averages approximately 240 deg. F. This change in operation will naturally effect a saving in coal. The entire cost of the auxiliary equipment was $43,873.54.

C. F. Goob is chief engineer of the Department of Public Works. The pumping station is under the general supervision of Milton J. Ruark, sewerage engineer, and the writer.

Shaking Table for Earthquake Tests

At Stanford University, Palo Alto, Calif., a shaking table has been built to experiment on models of buildings and other structures with respect to earthquake resistance and bracing. It consists of a platform of I-beams bolted and welded together, about 10x12 ft., weighing 6,000 lb., or, with attachments, about 8,000 lb. It rests by ball bearings on four railway wheels ground to true circular form, placed on short lengths of rail. Three helical springs with stiffness of 4,000 lb. per inch, anchored to concrete piers, cushion the longitudinal motion of the table. Two forms of drive are provided. To produce undamped harmonic motion a vertical flywheel with adjustable unbalance, belt driven by an electric motor, can be mounted rigidly on the table. To produce damped vibrations set up by a sudden force application there is an 1,100-lb. pendulum 17 ft. long, arranged to strike a bumper spring fastened to one end of the table. Mathematical studies of the types of vibration of the table, checked by measurement, are given by L. S. Jacobsen in the March, 1929, issue of the *Bulletin* of the Seismological Society of America.

Measuring Strain and Temperature in a 160-Ft. Concrete Arch Bridge

Tests Carried Out by Aid of Electric Telemeters Attached to Both Concrete and Steel—Action of Arch Rib as Dead Load Is Progressively Applied During Construction

By Searcy B. Slack
Bridge Engineer, State Highway Board of Georgia, East Point, Ga.

PRACTICALLY all tests and measurements for determining the behavior of reinforced-concrete arches have been made on completed structures, either full size or model. The measurements were made of strain due to varying loads or changes in temperatures, the increase or decrease beginning at some unknown initial strain caused by dead load, temperature changes and setting phenomena of the concrete.

An ideal set of stress measurements for an arch rib should show the total stress in both steel and concrete at the critical sections, beginning at zero stress when the concrete has just been placed and progressing through the setting stage, the striking of centering, the construction of the superstructure and finally the application

FIG. 1—CONCRETE ARCH OF 160-FT. SPAN UPON WHICH TESTS WERE MADE

of the live loads, including the variations due to changes in moisture conditions and temperature. Such an ideal set of stress determinations cannot perhaps be attained with present methods based on measurements of strain and conversion of these into stresses by multiplying by the modulus of elasticity, because in the early stages of the work or over a long period of time there is much evidence to show that strains in concrete are not directly proportional to stress.

However, much can be learned from measurements during the early stages of construction of concrete arches, and a series of measurements and tests was undertaken with this idea in view during the construction of a reinforced-concrete arch bridge located in Putnam County near Eatonton, Ga. The principal measurements and tests included: (1) concrete control to determine crushing strength and modulus of elasticity of the concrete at various ages; (2) measurements of strain in both concrete and steel at three sections of the arch rib, measurement beginning just after the concrete was placed; (3) measurement of vertical rib movement, begining just after the rib was completed; and (4) temperature measurements of concrete and steel throughout the rib.

Description of Project—The bridge on which the measurements were made (Fig. 1) is a two-rib open-spandrel arch with a span of 160 ft. and a rise of 46 ft., with concrete girder approach spans at either end. A general elevation and plan of the structure are shown in Fig. 2. The abutments at each end of the arch span rest on ledge rock, and the centering for the arch rib, consisting of ordinary wood falsework at the ends with three latticed steel truss spans for the central section, was also founded on rock. Allowance for settlement and deflection was made in setting the forms.

All materials for the bridge were handled by an overhead cableway located over the center line of the structure. The arch ribs were divided into five sections by radial bulkheads and concrete was placed in both ribs at the same time; the crown section was completed first, followed by the haunch sections, and these by the two sections between. After the centering was lowered, construction of the spandrel columns and decks progressed as indicated in Fig. 5.

Concrete Control Specimens—The control specimens of the concrete were made regularly while concrete was being placed in the arch ribs and were stored in damp sand and tested in the highway department laboratory. The 6x12-in. cylinders showed an average 28-day strength of 3,250 lb. per sq.in. at 28 days, increasing to 4,763 lb. at 90 days. The modulus of elasticity test, made on 6x12-in. cores drilled from 12-in. test cubes, showed a value of 4,400,000 at 28 days, which increased to 5,280,000 at 90 days. The crushing strength of these cores ranged from 2,525 lb. per sq.in. at seven days to 3,095 lb. at 28 days and 4,560 lb. at 90 days.

To check zero readings of the instruments and the thermal coefficient of the concrete, a test block, 24 in. wide, 22 in. deep and 72 in. long, was cast. It was reinforced with ten ¾-in. diameter bars hooked at each end, five bars being placed near the top face and five bars near the bottom face. Since it was intended to have no stress in this test block, the forms were left on the concrete and the supports were not removed.

Strain Measurements—All strain measurements in both steel and concrete were made by means of McCullom-Peters electric telemeters, a type of electrical strain gage. The single-resistor cartridge type was used in the concrete and the double-resistor type was used on the steel and the surface of the concrete. (A full description of these instruments was published in the 1927 *Proceedings* of the A.S.T.M., in a paper by O. S. Peters, an abstract of which paper appeared in *Engineering News-Record*, July 7, 1927, p. 25.) In these instruments, changes in length between gage points are measured by changes in resistance of a carbon stack, the length of which is varied by strain in the material.

The telemeters on the surface of the concrete were fastened by screws to short steel plugs which were placed in the concrete while it was still plastic. The telemeters on the steel reinforcing bars were fastened by screws

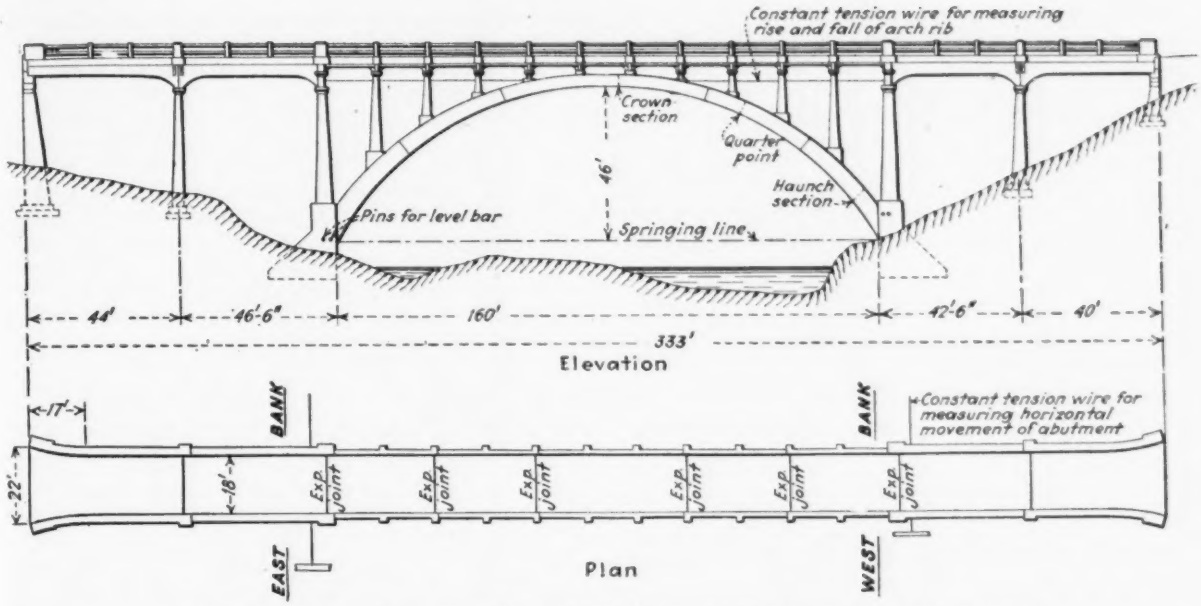


FIG. 2—PLAN AND ELEVATION OF ARCH UPON WHICH MEASUREMENTS WERE MADE

Note wire for measuring rise and fall of ribs. Telemeters were installed on steel and concrete at both the extrados and intrados of the crown section, the quarter point and the haunch section, and cartridge type telemeters were installed at the same sections within the concrete.

into tapped holes in the bars. Short sections of the bars to which the telemeters were to be fastened were left exposed by building small boxes around them to keep out the wet concrete, as shown in Fig. 3. The cartridge-type telemeters were fastened in place in the forms by wire before the concrete was poured.

The telemeters were placed on the west half of the upstream arch rib and were located so as to measure strains at the crown section, at the quarter point and at a section near the haunch; also on the test block. They were fastened to the surface of the concrete and to the steel bars at both intrados and extrados at all three sections, and to the surface of the steel both top and bottom of the test block. The cartridge-type telemeters were placed in the ribs at the crown and haunch sections and at the center of the test block. All lead wires from the telemeters were carried to the instrument house, where readings were made.

Resistance thermometers were fastened to each of the telemeters, since in interpreting readings it was necessary to make corrections for temperature changes, as there is a difference in temperature between the instrument and the concrete and also a difference in the coefficient of expansion of the instruments and the concrete.

The telemeters were fastened on the steel bars as soon as possible, readings being taken 24 hours after the concrete was placed. Two readings were taken each day, one set at 6 a.m., when the temperature was very near minimum, and the other set at 3 p.m., when the temperature was very near maximum.

Test Block "Stress"—It was intended to use the readings from the telemeters fastened to the test block as checks, since there was no known stress to be expected in the test block except very minor bending stresses. The actual readings, however, indicated considerable stress. It was at first believed that these surprising readings were caused by errors in the instruments or methods of reducing the readings, but a careful check showed that the instruments were functioning properly.

After field work on the arch rib had been completed, it was decided to make another set of measurements on two new test blocks of about the same size and reinforced

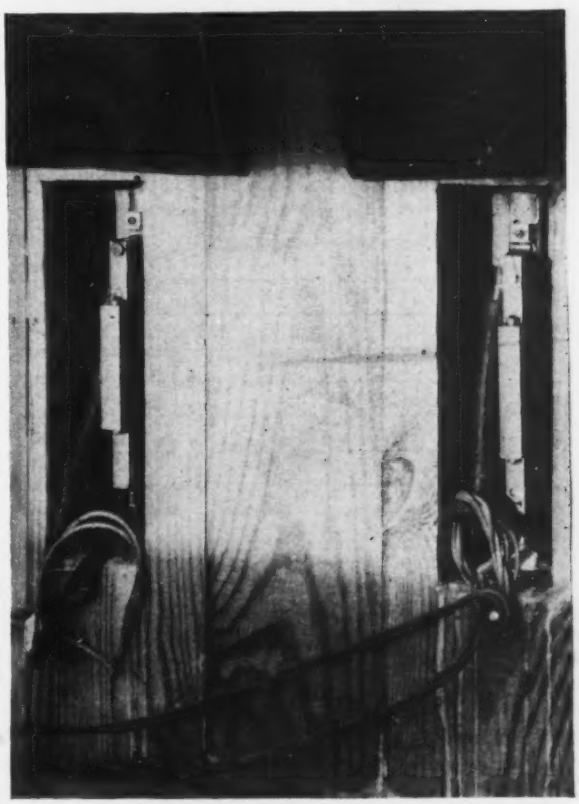


FIG. 3—METHOD OF ATTACHING TELEMETERS AND ELECTRIC THERMOMETERS TO REINFORCING STEEL

Boxes were built in the forms, shutting off the concrete from a short section of reinforcing bar. Canvas covers for the boxes are shown lifted in the illustration.

in the same way as the original test block. Four telemeters with resistance coil thermometers were fastened to each block, and a record of the daily maximum and minimum air temperatures of the concrete showed the same characteristic early rise and gradual fall as was found in the concrete of the arch rib.

The stress developed in the steel of one of the test blocks varied over a period of about a month between 2,000 and 3,000 lb. per sq.in. Part of this tension may be explained by the difference in the coefficient of expansion between these points being measured with a Whittemore strain gage. The bars were then cut at a point just above the upper gage point, thus releasing all stress in the bars between gage points without disturbing the points. The average stress as measured in this way was 3,000 lb. per sq.in., as compared with 3,300 lb. per sq.in. as shown by the telemeters.

The curves of Fig. 4 are typical of the 23 stress curves which were plotted by computing stresses from the strain measurements. They represent averages of strain

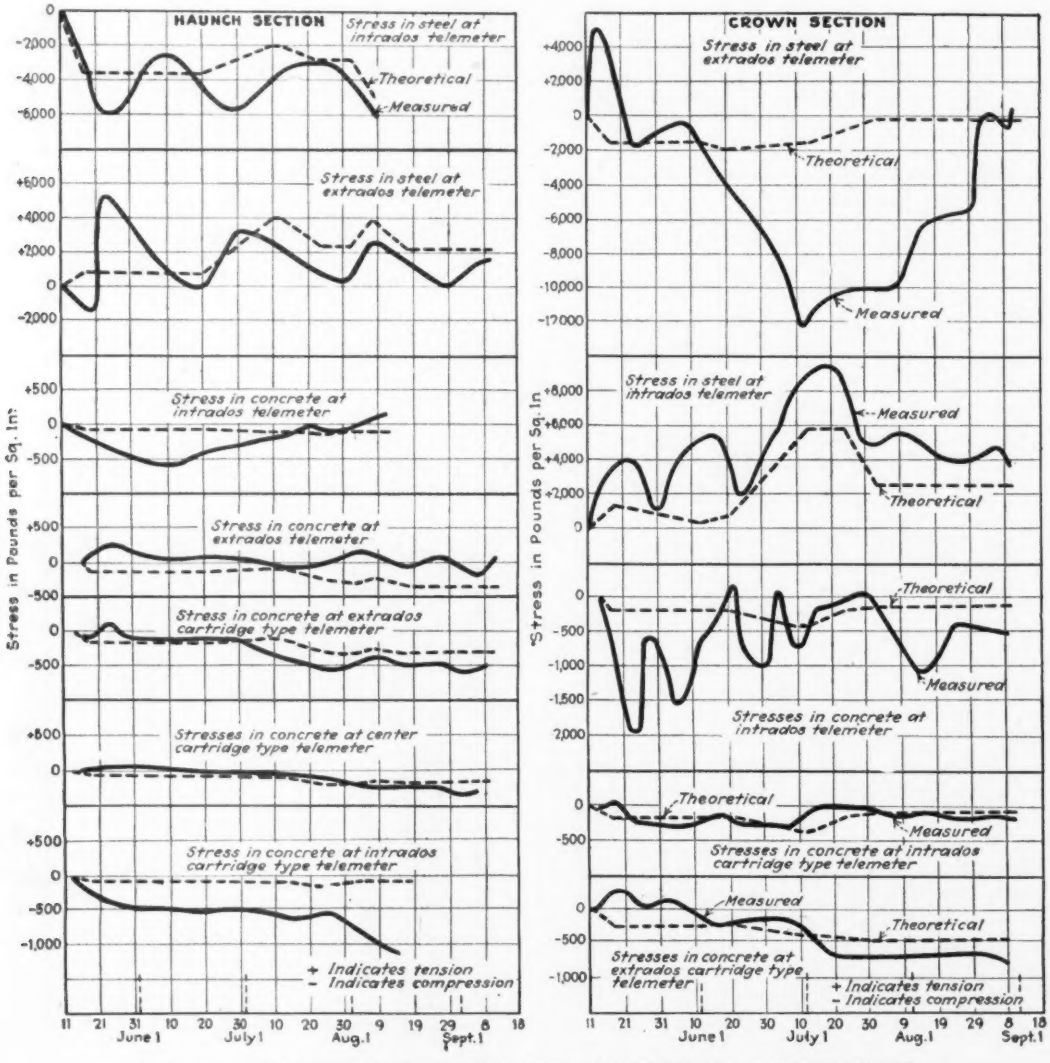

HAUNCH SECTION
Stress in steel at intrados telemeter
Theoretical
Measured
Stress in steel at extrados telemeter
Stress in concrete at intrados telemeter
Stress in concrete at extrados telemeter
Stress in concrete at extrados cartridge type telemeter
Stresses in concrete at center cartridge type telemeter
Stress in concrete at intrados cartridge type telemeter
+ Indicates tension
- Indicates compression
Stress in Pounds per Sq. In.
CROWN SECTION
Stress in steel at extrados telemeter
Theoretical
Measured
Stress in steel at intrados telemeter
Measured
Theoretical
Theoretical
Measured
Stresses in concrete at intrados telemeter
Theoretical
Measured
Stresses in concrete at intrados cartridge type telemeter
Measured
Theoretical
Stresses in concrete at extrados cartridge type telemeter
+ Indicates tension
- Indicates compression
Stress in Pounds per Sq. In.
11 21 31 10 20 30 10 20 30 9 19 29 8 18
June 1 July 1 Aug. 1 Sept. 1

FIG. 4—TYPICAL STRESS CURVES PLOTTED FROM STRAIN-GAGE READINGS
Note that measured stress is in general greater than the theoretical stress shown by the dotted curves.

of steel and concrete. Assuming the coefficient of expansion of steel to be 0.0000117 and the coefficient for concrete to be 0.0000082, the difference in the coefficients is 0.0000035. The fall in the temperature of the concrete of the test block was 17 deg., so that the approximate stress in the steel was $0.0000035 \times 17 \times 30{,}000{,}000 = 1{,}785$ lb. per sq.in. Stress in the steel of one of the test blocks as measured was 3,300 lb. per sq.in. at one time, which is seen to be more than can be accounted for by the difference in the coefficients of expansion. To check the stress in the steel, the telemeters were removed and gage points were established on the bars, the distances measurements and trends as shown by two sets of readings each day. Erratic readings have been ironed out.

On each of the curves the theoretical stress represents dead load only, no allowance being made for temperature, rib shortening or shrinkage. The stresses were computed on the basis of the steel taking all tension, an assumption generally made but which is probably in error. The changes in the dead-load stresses were caused by varying loads as the construction of the spandrel columns, floorbeams and decks progressed. The curves show that in general the measured stresses are higher than the theoretical stresses. The strains measured on

the surface are much more erratic than the strains measured by the cartridge telemeters embedded in the concrete.

Vertical Movement of the Arch Rib—The rise and fall of the arch ribs at the crown section was measured from a tight steel wire fastened to the tops of the columns over the abutments as shown in Fig. 2. An engineer's scale was fastened on the face of the arch rib so that vertical movement could easily be read.

The readings indicated a steady fall in the arch rib during the first few days after concrete was placed. At the end of the third day there was a rise and fall in the arch rib as shown by morning and afternoon readings, and this daily rise and fall due to differences in temperature between night and day continued (Fig. 5).

As any movement of the abutments would have seriously affected stresses in the arch rib and also the rise and fall of the arch rib at the crown, a careful check was made on the abutments for both angular rotation and lateral movement. The measurements made did not indicate any movement in the abutments.

Temperature Measurements—The temperatures at various points throughout the arch rib and at each of the telemeters were measured twice each day by the resistance coil thermometers. A record was also kept of maximum and minimum air temperatures each day. A study of the curves in Fig. 6 shows a characteristic rise within a few hours after concrete had been placed. Following this initial rise, the temperature remained practically stationary for several days and then gradually fell until the temperatures in the concrete were practically the same as the mean air temperature. A comparison of results of these measurements with temperature studies which have been made at other arch bridges and in several large dams leads to the conclusion that there is probably a variation in the initial rise of temperature for different brands of cement, and further that the mass of the concrete largely determines the time during which the temperature in the concrete remains at a high level.

The arch rib developed sufficient strength during the first three or four days to begin rising and falling with a daily variation in temperature. At this time the internal temperature of the concrete was still quite high, and this temperature fell rapidly after the rib began functioning as an arch. From this it must follow that temperature stresses are based upon strains caused by fall in temperature only. At no period of its life will the temperature inside the arch rib reach the high levels caused by the set in the cement. It thus appears that

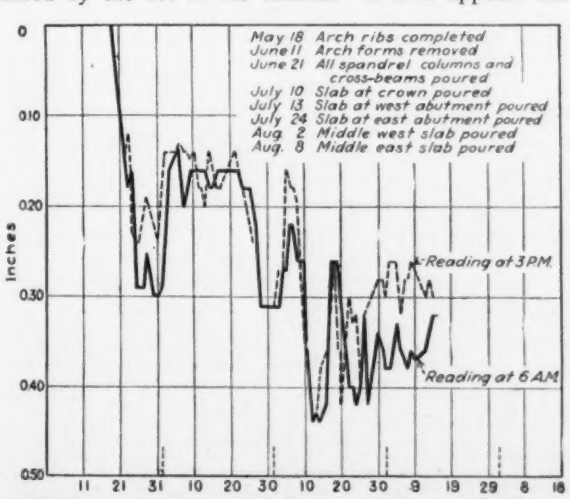

May 18 Arch ribs completed
June 11 Arch forms removed
June 21 All spandrel columns and cross-beams poured
July 10 Slab at crown poured
July 13 Slab at west abutment poured
July 24 Slab at east abutment poured
Aug. 2 Middle west slab poured
Aug. 8 Middle east slab poured
Reading at 3 P.M.
Reading at 6 A.M.
Inches
0
.010
.020
.030
.040
.050
11 21 31 10 20 30 10 20 30 9 19 29 8 18
May 2 June 1 July 1 Aug. 1 Sept. 1

FIG. 5—DAILY RISE AND FALL OF ARCH RIBS

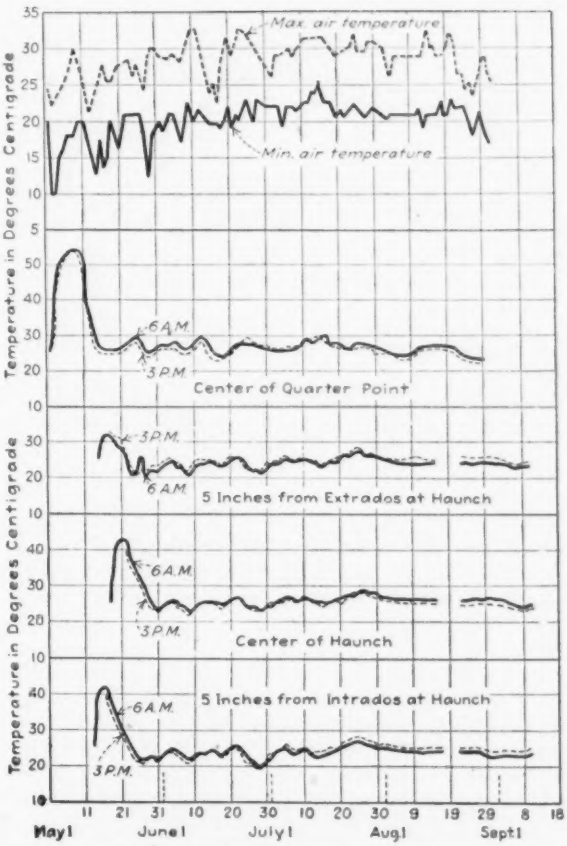

Max. air temperature
Min. air temperature
Temperature in Degrees Centigrade
6 A.M.
3 P.M.
Center of Quarter Point
5 Inches from Extrados at Haunch
Center of Haunch
5 Inches from Intrados at Haunch
11 21 31 10 20 30 10 20 30 9 19 29 8 18
May 1 June 1 July 1 Aug. 1 Sept. 1

FIG. 6—CONCRETE TEMPERATURES
Note comparison with daily maximum and minimum air temperatures during the test, shown in the curve at the top.

strains due to temperature in an arch rib should be based entirely upon a fall of temperature rather than the usual practice of assuming a rise and fall. It is quite likely that there is some readjustment in the concrete after the initial fall in temperature, but it is hardly possible that this adjustment would entirely relieve strains caused by the large initial drop in temperature.

Conclusions—It is recognized that the measurements made were not conclusive, but they indicate phenomena worthy of further study. The following conclusions, however, seem to be justified:

1. The observations indicate that the arch ribs began rising and falling with varying atmospheric temperature between the third and fourth day after the concrete work had been completed.

2. Due to set of concrete while initial temperature is high, the allowance made for temperature stress in the design of concrete arches should be based entirely upon a fall of temperature, and no allowance should be made for rise in temperature.

3. The observed stresses in the steel and concrete do not check with the computed stresses. The observed stresses were consistently higher.

4. The observations indicate that in reinforced concrete there are very appreciable initial stresses developed in both steel and concrete, probably due to setting phenomena of the concrete.

The tests here presented were made by the State Highway Board of Georgia under the direction of the writer. Valuable assistance in this work was given by the U. S. Bureau of Public Roads, the U. S. Bureau of Standards and the American Society of Civil Engineers.

Analysis of Dynamic Stresses in Hackensack Bascule

Study of Design Based on Spring-Acting Flexible Counterweight Tower—Results Compared With High Stresses Found by Investigators

By C. E. Paine
Vice-President, Strauss Engineering Corporation, Chicago

THE FINAL report of the board of engineers appointed to investigate the cause of failure on Dec. 15, 1928, of the east bascule leaf of the Hackensack River bridge on the Lincoln Highway near Jersey City, N. J. (*Engineering News-Record*, June 6, 1929, p. 916), places the cause of the failure in the inadequacy of the design of the counterweight tower. The board supports its finding by citing calculated unit stresses at the base of the tower caused by the deceleration of the counterweight when the moving leaf in the fully open position is stopped by application of the brakes. It also shows, combined with calculated unit stresses, a unit stress determined by field measurements of oscillatory vibrations of the tower posts of the west leaf.

The unit stresses computed by the board for various combinations of braking forces, wind and friction are of startling magnitude, and coming as they do from a board of experts, their correctness ordinarily would not be questioned except by those experienced in the design of similar elastic structures. In view of the apparent interest in such structures and in order to correct the impression that proper attention is not given to dynamic effects in the design of bascule bridges, it seems desirable to present a simple and practical solution.

Analysis

Description of Action—When the bridge is operating normally, the leaf starts from rest with an acceleration approximately uniform and in a short interval of time it attains a uniform velocity corresponding to the speed of the motors at the torque required to overcome the resistances of wind and friction. The counterweight is not accelerated as quickly as the leaf except when movement is started with the link in a horizontal position. While the vertical component of the counterweight velocity is always equal to the vertical component of the counterweight trunnion velocity, the horizontal components are not equal during periods of acceleration or deceleration. During acceleration of the leaf the acceleration of the counterweight in the horizontal direction is effected by building up a link stress of sufficient magnitude through horizontal displacement of the tower post. This action will start an oscillation of the counterweight which will be promptly damped out by friction at the counterweight trunnion, after which the counterweight and counterweight trunnion will move with the same uniform velocity so long as the velocity of the leaf is uniform.

If, while the bridge is being opened, power is shut off from the motors and the brakes are applied with wind and friction resisting motion, the structure will be stopped in two principal steps—first the leaf will be stopped and then the counterweight. The deceleration of the leaf will be nearly uniform due to the combined resistance of wind, trunnion friction and mechanical braking. The deceleration of the counterweight depends upon counterweight trunnion friction and a resisting force built up in the links by deflecting the tower post. Hence this deceleration will vary with the horizontal displacement of the counterweight with respect to the counterweight trunnion. The counterweight will swing forward until its velocity with increasing deceleration has been reduced to zero, after which it will swing back again and continue to oscillate until its movement has been damped out by friction.

Deceleration of the Leaf—Neglecting backlash between gears and the torsional displacement of shafts, it is sufficient to add to the kinetic energy of the moving leaf the kinetic energy of the counterweight, thus obtaining the total kinetic energy which must be overcome while stopping the leaf by the combined action of wind, friction and brakes. It will be seen later that the error is negligible if we include here with the leaf only that part of the kinetic energy of the counterweight which is due to the vertical component of its velocity. From these data the time of stopping the leaf may be computed. Within reasonable limits, it makes little difference what this time is, as will be shown later.

To consider a specific case: When stopping the Hackensack bascule, with the leaf moving at full speed in the fully open position (86 deg.), by instantaneous application of motor brakes and emergency brakes and with an assumed trunnion friction of 18 per cent and a 15-lb. wind opposing the motion, it will be seen from Fig. 2 that the direction of motion of the counterweight and

BASIC DATA FOR HACKENSACK BASCULE DESIGN

(All figures are given for one-half of one leaf)

Kinetic Energy of Moving Parts, Motor at 840 r.p.m.

	R.P.M.	K.E., Ft.-Lb.
Moving leaf (exclusive of counterweight)	0.169	4,700
Machinery (inc. emergency brake)	.....	1,774
Motor rotor, pinion and brake drum	840	4,200
		10,674

Counterweight velocity = 0.326 ft. per second. K.E. = 1,300 ft.-lb.

Friction Torques

	Dia., Ft.	12 Per Cent Friction, Ft.-Lb.	18 Per Cent Friction, Ft.-Lb.
Main trunnion	1.375	93,300	140,000
Ctwt. trunnion	2.25	106,700	160,000

Motor Brake Torque, 250 ft.-lb. at 840 r.p.m., allowing 10 per cent for machinery losses, is equivalent to 1,370,000 ft.-lb. applied at main trunnion.

Emergency Brake Torque, 2,000 ft.-lb. at 146.5 r.p.m., allowing 10 per cent for machinery losses, is equivalent to 1,906,000 ft.-lb. applied at main trunnion.

Wind on Leaf and Counterweight (Angle of opening 86 deg.) 15-lb. wind.

Net moment about trunnion = 2,109,000 ft.-lb.

Wind on counterweight = 7,320 lb. at 27 ft. above counterweight trunnion.

Elastic Relations

Bridge open 86 deg.: See Fig. 3.

Bridge closed:

1,000 lb. applied as in Fig. 3 will cause a horizontal displacement of the counterweight equal to 0.0159 ft., a link stress having a horizontal component of 1,077 lb. and horizontal and vertical displacements of the second link pin equal to 0.0218 and 0.00183 respectively.

A compressive link stress having a horizontal component of 1,000 lb. will cause tensile unit stresses at the base of the tower post equal to 2,369 lb. per sq.in. for 86 deg. opening; 2,187 lb. per sq.in. for 79 deg. opening; and 1,639 lb. per sq.in. for bridge closed.

counterweight trunnion is normal to the link. The velocity is 0.326 ft. per second, the horizontal component of the velocity is 0.313 ft. per second and the vertical component is 0.092 ft. per second. That part of the kinetic energy due to the vertical component of the velocity is 104, which, added to 10,674 (the kinetic energy of the other moving parts), gives a total of 10,778. The combined effect of wind, friction and brakes is equal to 5,685,000 ft.-lb. (see data in table) applied at the main trunnion. The angular displacement of the leaf during deceleration $= 10{,}778 \div 5{,}685{,}000 = 0.0019$ radians. Initial velocity $= 0.169 \times 2\pi \div 60 = 0.0177$ radians per second. Hence, time of deceleration of counterweight trunnion $= 0.215$ second. Deceleration of counterweight trunnion $= 0.326 \div 0.215 = 1.515$ ft. per second per second.

Wind and Friction—Wind on the leaf tends to oppose its movement, whereas wind on the counterweight reacts

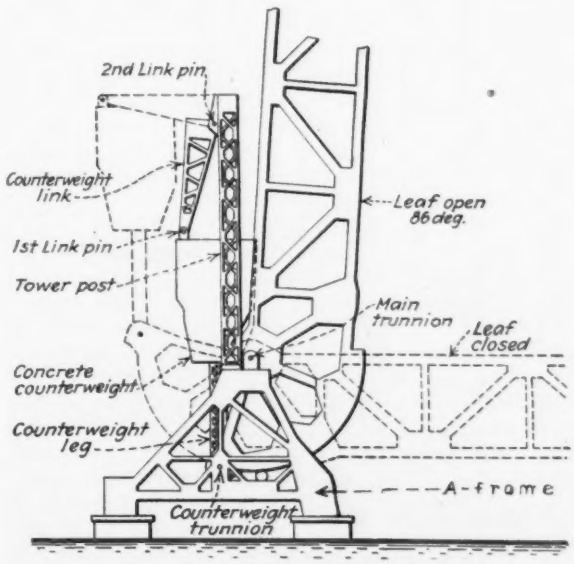


FIG. 1—RELATIVE POSITIONS OF LEAF AND COUNTERWEIGHT WITH LEAF OPENED AND CLOSED—HACKENSACK BASCULE BRIDGE

to aid its movement. The net result can be expressed as a retarding moment about the main trunnion which increases the deceleration of the leaf. The wind has no effect on the deceleration of the counterweight except indirectly as it affects the deceleration of the leaf.

Friction at the main trunnion and counterweight trunnion opposes the movement of the leaf and thus affects its deceleration. At the time deceleration starts the portion of the link stress due to trunnion friction has a moment about the counterweight trunnion equal and opposite to the friction moment. Hence, if we retain this portion of the link stress constant throughout the deceleration of the counterweight, no further consideration need be given to counterweight trunnion friction in so far as deceleration is concerned. It should be noted, however, that when the counterweight reaches the end of its swing and motion reverses, friction will also reverse and damp the vibration.

Therefore, in the calculations wind and friction should be included in determining the deceleration of the moving leaf but need not be further considered in calculating stresses due to deceleration of the counterweight. Stresses in the tower post due to wind and friction should be algebraically added to the stress caused by deceleration of the counterweight.

Deceleration of Counterweight—The forces causing deceleration of the counterweight with reference to its trunnion may be found either by direct computation for successive time intervals or by solution of differential equations.

At the beginning of deceleration of the counterweight trunnion, it is moving with the same velocity as the counterweight. During the first small interval of time succeeding application of the brakes, the trunnion is decelerated, but the counterweight moves with the same velocity as before, because the tower post has not been displaced and no decelerating force has been induced in the link.

At the end of this first interval the counterweight is displaced with reference to the trunnion and there is a deflection of the tower post which induces a force in the link and a corresponding deceleration of the counterweight; this induced force can be computed from the data given in Fig. 3.

During the next interval the trunnion moves with a velocity reduced by its deceleration during the first interval, and the counterweight moves with its initial velocity reduced by the decelerating effect of the stress in the link.

In succeeding intervals the counterweight gains further on the trunnion, and the displacement of the tower, the stress in the link and the deceleration of the counterweight are all successively increased. After the trunnion comes to rest, the counterweight gains further on the trunnion with increasing link stress and deceleration. Finally the decelerating force becomes so great that the counterweight stops. This is the instant of maximum deflection and hence of greatest unit stress in the tower post.

Differential Equation of Deceleration—The relation between decelerating force and deceleration is expressed mathematically as $F = -Ma$, in which F is the horizontal component of the resultant of that portion of the link stress and trunnion reaction which produces deceleration a of the counterweight, the mass of which is M.

All displacements y and velocities V are measured horizontally with reference to a line through the counter-

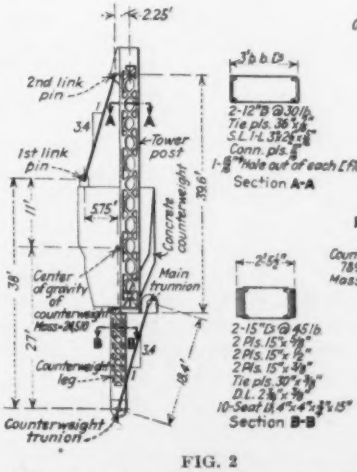


FIG. 2

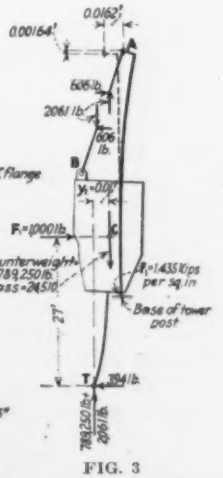


FIG. 3

Fig. 2—Arrangement of counterweight system, Hackensack bascule bridge. As shown, leaf is open to an angle of 86 deg. above the horizontal; counterweight in lowest position. Fig. 3—Elastic relations of counterweight system. Angle of leaf opening 86 deg. Figures include secondary effect due to displacement of the counterweight with respect to counterweight trunnion.

weight trunnion parallel to the line connecting trunnion and center of mass of counterweight before brakes are applied (Fig. 3).

After brakes are applied the trunnion has, until it stops, an acceleration a_t, which we shall assume as constant. Then

$$a = \frac{d^2y}{dt^2} - a_t$$

$$F = y\frac{F}{y} = -M\left(\frac{d^2y}{dt^2} - a_t\right)$$

$$y = -M\frac{y}{F}\left(\frac{d^2y}{dt^2} - a_t\right)$$

Since the physical properties of the structure determine a constant relation between force and deflection, let $M\frac{y_1}{F_1} = b$, where y_1 and F_1 are as given in Fig. 3.

Then $y = -b\left(\frac{d^2y}{dt^2} - a_t\right)$, which is satisfied by

$$y = a_t b\left(1 - \cos\sqrt{\frac{1}{b}}\,t\right) \qquad (1)$$

since velocity and displacement are both zero at the beginning.

Equation 1 governs until the trunnion comes to rest, after which $a_t = 0$, and $y = -b\frac{d^2y}{dt^2}$ which is satisfied by

$$y = y'\cos\sqrt{\frac{1}{b}}\,t + V''\sqrt{b}\sin\sqrt{\frac{1}{b}}\,t \qquad (2)$$

if, when $t = 0$, the relative velocity of the counterweight is V'', and the relative displacement is y'.

Determining the maximum value of y from equation 2 and substituting values of V'' and y', determined from equation 1 for $t = t_o$ (the time when the trunnion stops), and for a_t the value $\frac{V_o}{t_o}$ where V_o is the velocity before brakes are applied, we obtain

$$y_{max.} = V_o b\sqrt{2}\,\frac{\sqrt{1 - \cos\sqrt{\frac{1}{b}}\,t_o}}{t_o}$$

This expression gixes maximum possible displacements due to deceleration for different times of stopping the leaf up to $t_o = \pi\sqrt{b}$, after which the maximum is determined from equation 1 and varies inversely as t_o. The time interval between maxima, or natural period of vibration, is

$$2\pi\sqrt{b} = 3.12 \text{ seconds}$$

The maximum possible value of y occurs when the leaf is stopped instantly, and is obtained from equation 2 for $y' = 0$ and $V'' = V_o$, as absolute maximum $y = V_o\sqrt{b} = 0.155$ ft.

Whence absolute maximum unit stress at base of tower post due to deceleration $= 0.155 \times 1{,}435 \div 0.01 = 22{,}200$ lb. per square inch. The same value may be obtained by equating the loss of energy to the work done when the trunnion is stopped instantly.

The coefficient $\frac{\sqrt{1 - \cos\sqrt{\frac{1}{b}}\,t_o}}{t_o}$

is almost constant within the operating range as t_o varies.

Hence the maximum stress in the tower post due to deceleration is nearly independent of the time of stopping the trunnion and the rate of deceleration of the leaf. These stresses can be accurately and easily determined by applying elementary principles of mechanics with or without the aid of calculus.

Discussion of Analysis—The curve in Fig. 4 shows the maximum unit stress in the tower post resulting from stopping the leaf in different periods of time. It is noticeable that it makes very little difference whether the motor brakes only are used or whether both motor brakes and emergency brakes are applied simultaneously; the rate of stopping the trunnion is not the important factor in this problem. The determining factors are: the mass and initial velocity of the counterweight, the stiffness of the tower post and the geometric relations of the various elements of the structure at the time deceleration starts.

Maximum Stress Conditions

Operating Conditions—The Hackensack bridge was designed to be operated against a wind pressure of 7½ lb. per square foot. It could not be operated against a

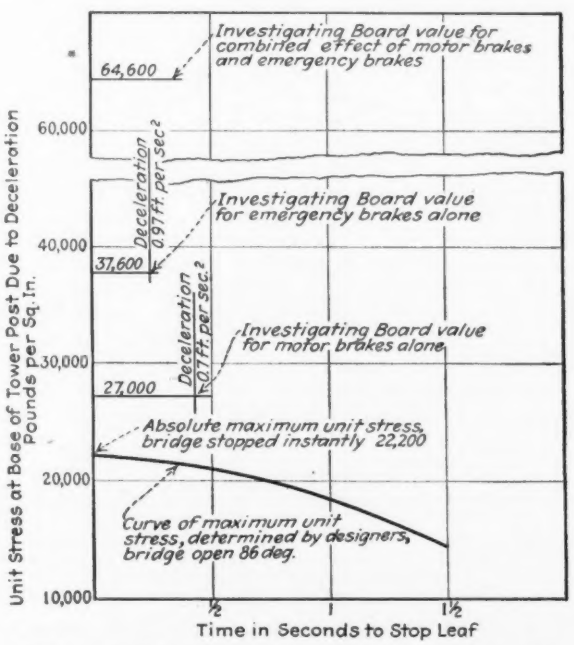

Investigating Board value for combined effect of motor brakes and emergency brakes
64,600
60,000
Deceleration 0.97 Ft. per Sec.²
Investigating Board value for emergency brakes alone
40,000
37,600
Deceleration 0.71 Ft. per Sec.²
Investigating Board value for motor brakes alone
30,000
27,000
Absolute maximum unit stress, bridge stopped instantly 22,200
20,000
Curve of maximum unit stress, determined by designers, bridge open 86 deg.
10,000
Unit Stress at Base of Tower Post Due to Deceleration Pounds per Sq. In.
½
1
1½
Time in Seconds to Stop Leaf

FIG. 4—UNIT STRESSES IN TOWER POST DUE TO DECELERATION OF COUNTERWEIGHT

wind pressure of 15 lb. per square foot, and in order to get a combination of the decelerating force with a 15-lb. wind it would be necessary to assume that the wind suddenly increased to that pressure simultaneously with the application of the brakes.

Protecting Devices—The bridge is equipped with limit switches which automatically cut off power from the motors and apply the motor brakes when the leaf in opening reaches an angle of opening of 79 deg. Power to open the leaf beyond this point can be obtained only by repressing a bypass switch which will permit the application of a limited amount of power to complete the movement after passing the 79 deg. position. With a 7½-lb. wind opposing movement it would be impossible to get the bridge up to anywhere near its full speed and probably not more than half speed after passing the 79 deg. position.

Combining Calculated Stresses—The following combinations of stresses at the base of the counterweight

post obtained under conditions assumed by the board are as follows:

	Correct Values	Board's Values
Due to deceleration	+22,200	+69,600
Due to 15-lb. wind	+14,000	+14,000
Due to 18 per cent friction	— 8,500	— 9,000
Total with wind	+27,700	+74,600
Total without wind	+13,700	+60,600

Mention was made in the report of vibratory stresses which were measured in the field. The procedure of measuring stresses in the field and adding these measured stresses to the computed stresses is unusual and questionable.

Effect of High Fluctuating Friction—Any acceleration or deceleration of the counterweight will be accompanied by a deflection of the tower post, thus imitating an oscillatory movement. Under circumstances within the range of normal operating conditions the oscillatory movement of the tower post and counterweight will be damped out quickly by trunnion friction. In a properly lubricated trunnion bearing, friction during motion should remain at a fairly steady value of 12 per cent with less than 3 per cent fluctuation. When the bearing is operated extensively without lubrication, not only does friction mount to a high figure but the movement becomes intermittent and jerky. Under such conditions friction fluctuates between an abnormally high value and an abnormally low value. It is evident that if maintenance of the structure is neglected to that extremity and operation is permitted under such circumstances vibrations of high amplitude might result. The counterweight trunnion bearing of the Hackensack bridge recovered from the river gives unmistakable evidence that it had been operated dry and with high fluctuating friction. Furthermore, the counterweight trunnions of the west leaf (which did not fail and on which the board's measurements were made) were operating in just such an abnormal manner. The effort to correct the condition by attempting to force grease into the grooves could hardly be expected to give good results, since no doubt the grease grooves needed to be thoroughly cleaned out in order to get new lubricant in and properly distributed over the surface of the trunnion.

Basis of Design—As a *normal* condition for operation the wind pressure is assumed to be 2½ lb. per square foot (about a 30-mile wind). The *extreme* operating condition is taken with wind pressure at 7½ lb. per square foot.

In all cases trunnion friction is taken at 18 per cent for starting and 12 per cent for moving. The investigating board reported an average trunnion friction of 13.3 per cent as determined by power measurements. The dependability of values found by such measurements is doubtful, because there are many other losses involved for which correction factors must be determined. At the best this gives only approximate *average* values which are no indication whatever of the value for any particular bearing nor of the maximum value where there is a great fluctuation due to improper lubrication.

The tower post is designed for a decelerating force due to stopping the bridge by the motor brakes at an angle of opening of 86 deg. with friction at 12 per cent and with a wind pressure of 2½ lb. per square foot opposing the motion. For the bridge stationary, in the closed position, the tower post is designed to withstand a wind pressure of 20 lb. per square foot on the counterweight and tower. The unit stresses at the base of the tower post for the above cases are as follows:

1. Stopping bridge in the fully open position while opening at full speed against a wind pressure of 2½ lb. per square foot and 12 per cent trunnion friction:

Deceleration of counterweight	+20,600 lb. per sq.in.
Wind	+ 2,300 lb. per sq.in.
Trunnion friction	— 5,700 lb. per sq.in.
Total	+17,200 lb. per sq.in.

2. Bridge closed: Wind pressure of 20 lb. per square foot on counterweight and tower.

Wind	+18,700 lb. per sq.in.
Friction at 12 per cent	—6,700* lb. per sq.in.
Total	+12,000 lb. per sq.in.

*Owing to its elasticity, the tower post will deflect and rotation will take place at the counterweight trunnion.

It is apparent that a certain degree of flexibility in the tower is desirable in order to absorb the inertia of the counterweight mass during periods of its acceleration or deceleration. The stress in the link and link pins is largely dependent upon the stiffness of the tower, and it follows that if the tower is made stiffer, the stresses in these parts will be increased accordingly.

Conclusions

The following conclusions are indicated:

1. The rate at which the leaf is stopped (deceleration of counterweight trunnion) has little effect on the maximum stresses in the tower post from deceleration.

2. The maximum stress computed by the board from deceleration is the greatest single factor in its amazing totals and is about three times the correct maximum value.

3. Maximum stresses for normal operation if computed by correct principles of mechanics are moderate.

4. Prolonged vibration during uniform movement indicates abnormal conditions in the bearings due to poor bushings and lack of lubrication.

5. The fact that the failure took place from fractures occurring at three different times indicates that it was due to the cracked flange rather than to general overstress. The original fracture at the rivet hole, in all probability, occurred during fabrication.

Proofs of the above article were submitted to the members of the official investigating board whose conclusions are traversed by Mr. Paine. Their reply follows.—EDITOR.

Investigating Board's Discussion of Counterweight Tower Action

IN CONNECTION with the collapse of the Hackensack River trunnion bascule bridge, it is significant that the designer's stress sheet does not show the stresses in the counterweight tower. It also is noteworthy that the accompanying specifications do not mention wind pressure, inertia forces or frictional resistance, although existing specifications of other authorship cover such subjects. That these forces are of vital importance in the design of the structure which supports the counterweight, which acts as an inverted pendulum, will be evident from the corrections to Mr. Paine's analysis made in the following paragraphs.

Unit Stresses

In order to indicate that the design of the tower does not conform to good practice, one needs only to discover stresses greater than allowable by any good specification.

The specification under which this bridge was designed allows tensile unit stress on net section of 16,000 lb. per sq.in. and compressive stresses on gross section of $16{,}000 - 70\ l/r$. In order to explain failure one must account for stresses greater than yield point of structural steel, say about 36,000 lb. per sq.in., at which value the counterweight tower channel will begin to stretch rapidly without further increase in stress until the counterweight tips forward and destroys the tower. It is important to note, therefore, that the yield point and not the ultimate strength of the steel governs; also that the major part of the stresses in the tower channel are completely reversible as load conditions reverse in direction, thus bringing into consideration compressive stresses and the endurance limit. If the errors in the application of the theory in Mr. Paine's article be corrected, the calculated stresses in the channel will become considerably greater than the yield point of the material.

Significance of Error in Period of Oscillation

Mr. Paine predicts an oscillatory period of 3.12 seconds as compared to an observed period at the bridge of 4.13 seconds. From his equation that the natural period of oscillation equals $2\pi\sqrt{b}$ it appears that the true value of $\sqrt{b}$ must be 4.13/3.12, or 1.32 times his estimated value. Furthermore, from his equation that maximum deflection, $y = V'_a\sqrt{b}$ and from the fact that deflection and stress are proportional, it follows that his estimated deceleration stress must be changed to 1.32 times 22,200, or 29,300 lb. per sq.in. One reason for this error is the assumption that the counterweight tower is fixed at its base. This clearly is incorrect, because the stresses in the A-frame caused by a load at the top of the counterweight tower cause movement of the top of the A-frame due to the elasticity of the members of the A-frame. This action was observed on the bridge.

Another reason for this error in estimated period is that the mass of the counterweight was considered concentrated at its center of gravity instead of at the center of percussion. Attention is also called to the neglect of the effect of the weight of the counterweight tower and half of the weight of the links and their bracing in modifying the oscillatory period, and the corresponding stresses and deflections.

Wind Stresses and Friction

We agree with Mr. Paine's estimate of 14,000 lb. unit stress due to a wind pressure of 15 lb. per square foot.

The most reliable measurement of friction in the counterweight trunnion bearings depends upon the measured change in stress in the channel of the counterweight tower as caused by the reversal of trunnion friction as the leaf is closed slowly and then slowly opened when there is no wind. This change of stress we charged to counterweight trunnion friction, and from these observations we compute a maximum coefficient of friction of 13.3 per cent. Our observations indicated that after a thoroughly fresh greasing of the counterweight trunnion bearing a coefficient of friction not exceeding 8 per cent was developed for a few movements of the leaf. Inasmuch as friction in a counterweight trunnion bearing reduces stress caused by other effects, this reduction should be estimated at the minimum observed value of 8 per cent and not at 18 per cent as estimated by Mr. Paine. This yields a correct deduction from unit stress on account of friction of 8/18(—8,500), or —3,800 lb. per sq.in.

Oscillation

Mr. Paine states in his article that the starting oscillation of the counterweight will be promptly damped out by friction at the counterweight trunnion. Contrary to this assumption, we have the remarkable phenomenon that an oscillating pendulum cannot be retarded by friction of the supporting shaft if the angular velocity of this shaft be greater than the maximum angular velocity of oscillation. This condition existed at the Hackensack bridge. On account of this phenomenon, which has been checked by laboratory experiments and other tests, the starting oscillation of the counterweight persists during the entire movement of the leaf as though the trunnion friction were zero, and furthermore this starting oscillation was observed to increase to double its initial value

NET UNIT STRESSES IN CHANNEL AT BASE OF TOWER POST, HACKENSACK BASCULE BRIDGE

Cause of Stress	Values by Paine	Value by Paine's Analysis as Corrected by the Board
Deceleration	+22,200	+29,300
Oscillation	Not given	+14,650
15-lb. wind	+14,000	+14,000
Friction	— 8,500 at 18 per cent	— 3,800 at observed 8 per cent
Total primary stress with wind	+27,700	+54,150
(Total primary unit stress without wind)	(+13,700)	(+40,150)
Secondary stress	Not included	+12,450 (23 per cent of primary)
Maximum unit stress	+27,700	+66,600
Yield point (unit stress corresponding to failure)	Not given	+36,000
Unit stress stated as allowable by design specifications	Not given	Tension on net sect. +16,000 Compression on gr. sect. 16,000—70 l/r or 12,465 lb. per sq.in. of channel section.

Note—The maximum stresses in this table were calculated on the theory that they would follow the same law above the yield point of the material which they are known to follow below the yield point. The calculated stresses above the yield point, therefore, are less than the probable stresses.

and to persist at its maximum value until the counterweight trunnion was stopped. If the brakes are applied at a time when the counterweight has its maximum oscillatory velocity, and in the same direction as the movement of the counterweight trunnion, the horizontal components of the two velocities should be added, thus increasing the value of V' which is used in calculating maximum displacement and the resulting stress.

The oscillatory effect adds considerably to other calculable stresses, and from our observations we find that a 50 per cent addition for oscillatory stress is a minimum permissible amount to add to the deceleration stress of 29,300 above.

Other Stresses

There are other possible causes of stress which are not calculable. The tower post is essentially a vertical truss with inclined diagonal members which do not intersect at the panel points. An estimate of the secondary stress caused by such eccentric details gave values about 23 per cent greater than the primary stress in the region where the failure occurred. Another possible cause of stress is eccentricity of the center of gravity of the counterweight due to its construction.

Summary

From the tabulated stresses it is apparent that a combination of braking stress, 29,300, oscillatory stress, 14,650, and a stress of 7,000 lb. per sq.in. due to a wind pressure of 7½ lb. per square foot, less a 3,800-lb. unit stress due to 8 per cent trunnion friction, gives a total of 47,150 lb. per sq.in. This is far above the yield point of structural steel and is adequate to account for unlim-

ited deflection of the tower which would mean failure.

Prolonged oscillation of the counterweight and tower during uniform movement of the counterweight trunnion does not indicate poor bushings or lack of lubrication, because this oscillation is independent of friction, as explained above. The chemical analysis of the bushings showed good material and there is little evidence that lubrication was neglected.

Poor lubrication should not wreck a movable bridge.

The fact that the fracture in the tower channel was progressive and occurred at three different times, as clearly shown by the appearance of the fracture, demonstrates that the failure was caused by recurring conditions of overstress.

The members of the board agree that in their opinion a tower which is depended upon to prevent the overturning of a heavy mass in unstable equilibrium should be designed and built to secure such rigidity that any perceptible oscillation cannot occur.

New York City,
Aug. 23, 1929.

O. E. HOVEY,
GEORGE E. BEGGS,
D. B. STEINMAN.

Glasgow's 720-Ton Refuse Incinerator

With Comparisons of British and American Refuse-Disposal Conditions

Garbage, Ashes and Refuse Produce Power and Concrete Aggregate—Metals Baled and Sold—Surplus Current Available After Serving Collection Wagons and Works Sold to City Electricity Department

By M. N. BAKER
Associate Editor, Engineering News-Record

HIGH-TEMPERATURE refuse destructors, with utilization of the heat of combustion for power and with the clinker residue used for road bottoming or aggregate for concrete, became common in Great Britain decades ago. In the United States and Canada this class of destructor was introduced considerably later. Heat utilization never gained much ground in the United States and for the most part was soon given up where attempted. It did better in Canada, notably at Toronto. No attempt to utilize the residue other than as filling seems to have been made on either side of the border. In Great Britain of late there has been a strong trend away from putting all the refuse through destructors. The older practice, but with a notable refinement, was recently adopted at Glasgow, where tailings of coarse material from screens are pulverized, then recombined with the fine screenings and the whole sent through a huge destructor plant, fully equipped with accessories to utilize both the heat and the clinker.

"Refuse utilization at Glasgow is carried to its greatest possible extreme."

Glasgow is operating the latest and in many respects most notable refuse-disposal plant in Great Britain. It is a high-temperature furnace, or destructor, and utilizes heat and clinker. Departing from recent British practice, it treats all the refuse, including garbage and ashes. Mr. Baker here tells of its operation, as seen on a recent visit, and adds illuminating comparisons between British and American conditions of refuse handling.

EDITOR.

At Glasgow early in July of this year I saw the new destructor, in operation about a year. Large as the plant is (640 long or 720 short tons per day), it does not serve the whole city, there being a number of old destructors still in use. Glasgow has about a million inhabitants.

The plant is located at Govan, on the south bank of the Clyde, on a site of 15½ acres. There are 40 cells, each of about 18 short tons daily capacity. The contract guarantees are based on one-fourth of the plant being out of service, and on operation six 24-hour days a week.

Refuse is collected and brought to the destructor in electric-propelled vehicles, charged by current produced by the plant. These vehicles are tipped by apparatus placed above the receiving hoppers instead of on the vehicles, thus reducing the vehicle weight. After being weighed, the refuse is hauled up a reinforced-concrete ramp and dumped into tipping apparatus, which in turn discharges into four receiving hoppers, one for each main group of furnaces. From the hoppers the refuse passes to revolving screens, which separate it *temporarily* into fine screenings and coarse tailings. A conveyor passes the coarse material beneath an electromagnetic separator, which removes "tins" and other metals. These are deposited in skips, taken to a platform and sorted into bright tins (for detinning) and galvanized and enameled tins. Both kinds are pressed into 12x12x18-in. bales, and sold—the bright tins for 35 and the others for 32½ shillings per long ton (about 3.8 and 3.5c. per pound).

Tailings, minus the metallic matter, pass through swinging-hammer pulverizers, then unite with the fine screenings, the whole going to the destructor furnaces. The hammers make 960 revolutions per minute and each is driven by a 35-hp. electric motor. The furnaces are charged by means of containers filled from the receiving hoppers, each feeding container being pushed over the feed hole by a hydraulic pusher. The furnaces are stoked and clinkered mechanically. The clinker is removed on a light railway and dumped into a pit, which may be flooded for quenching and subsequently pumped dry. From these pits the clinker is conveyed either to a crushing and grinding plant or to railway cars. The crushing and second screening plant discharges into large bunkers, from which the material is dumped into trucks or railway cars.

Some of the clinker is used for aggregate for curbing and sidewalk paving slabs. Some is sold to outsiders for making concrete. Apparently the fine screenings separated from the clinker are also used for concrete. As is the case in various other parts of the Glasgow

works, the clinker plant is constructed in duplicate.

Heat at 1,200 to 1,500 deg. F. from the combustion chambers of the destructors goes to water-tube boilers, which are provided with superheaters, automatic feed-water regulators and soot blowers for cleaning the tubes. The steam is used in two 5,000-kw. turbo-alternators.

Surplus current above works' needs, including the charging of the electric-collecting vehicles, is sold to the municipal electricity department at the cost of generating current at its light and power station. The actual price had not been settled when I was at Glasgow, owing to a disagreement between the cleansing and electricity departments over varying the price of the destructor current with the load factor, a point not covered when the electricity department agreed to take all the surplus current. Such interdepartmental differences in Great Britain go to the municipal council for settlement, if need be, and when sufficiently technical are referred to arbitrators. The various municipal departments are directed by large committees of council members which are sometimes overlapping in membership. This incident, coupled with the fact that the housing department, a large user of curbing and sidewalk slabs, had not, at the time of my visit, even inspected the concrete curbs and slabs put out by the cleansing department, suggests the need for more co-ordination between municipal departments than now exists—a need not confined to Glasgow. The Glasgow City Council has 110 members.

The buildings at the destructor plant are generally steel framed, with brick filling. Some of the interiors are finished with glazed brick. Floors are generally of concrete, finished with granolithic. Roofs are covered with asbestos sheets. Extensive foundation work was necessary in the way of concrete piles and mass concrete.

The destructor was designed and built under an inclusive contract by Heenan & Froude, Ltd., engineers and contractors, Worcester, England. There were about fifteen subcontractors. It cost some £600,000, or $2,920,000, against the original contract price of £437,000, or $2,127,000. Part of this excess of nearly 40 per cent was due to unexpectedly heavy foundation requirements.

Physically, the Glasgow destructor appeared to be working admirably. Chimneytops showed only a light vapor being discharged. Dirt, dust and odor were each as nearly absent as could be expected and were confined almost wholly to tipping operations at the refuse-receiving platform.

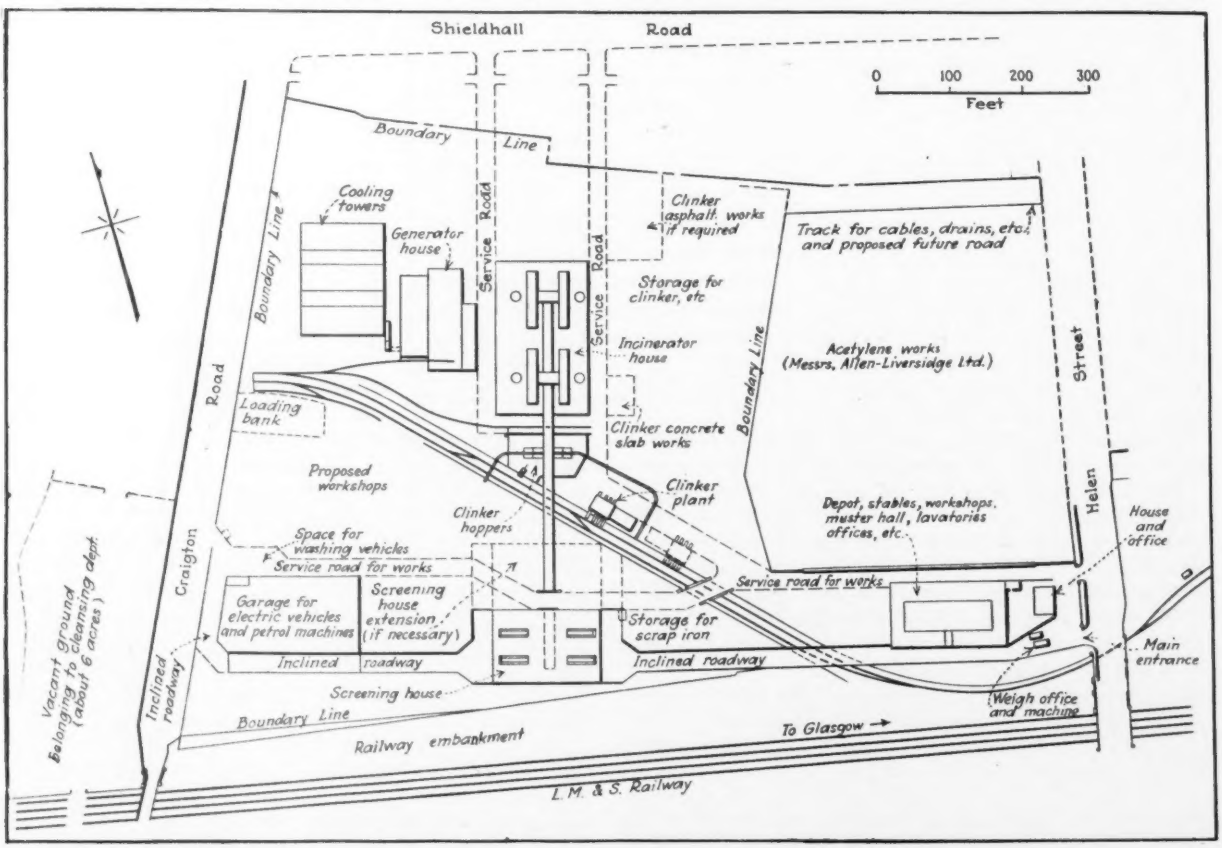
How the plant will work out as to yearly costs and revenues had not yet been determined when I was in Glasgow. Hope was expressed that when the plant has been fully adjusted revenue will equal the annual charges.

Wages at the plant are: lowest grade of labor, 9s. 2d., or $2.22; stokers, 10s. 2d., or $2.48; "lead" stokers, 10s. 5d., or $2.54; electricians and fitters, trade union rates of 11 and 12s., or $2.68 and $2.92. Slightly higher rates are paid for night work.

Colin H. Macfarlane is director and W. McArthur assistant director of cleansing of the city of Glasgow.

General Observations With Reference to British and American Conditions

Conditions governing refuse disposal are very different in Great Britain and America, not only as to the character of refuse and the physical possibilities of utilizing the heat and the furnace residue but also and particularly



GENERAL PLAN OF GLASGOW REFUSE DESTRUCTOR PLANT

as regards municipal administration. Moreover, although British municipal accounting is far ahead of American, yet I have seldom seen complete detailed figures of the expense and receipts of refuse disposal, taking capital charges into account. Such figures are necessary before one can judge what refuse disposal as practiced in Great Britain would cost in America. Months of detailed study, with the hearty co-operation of municipal officials, would be needed to get these figures in Great Britain, if indeed they could be obtained. It would be harder yet to obtain satisfactory figures on the net cost of refuse disposal in American cities.

Municipal administration is more favorable to utilization of refuse in Great Britain than in America. Politics affect municipal government far less there than here. That is true even in the election of councilmen. It is more true once they are elected. It is still more true of administrative staffs. Long tenure of office is common both among councilmen and their appointees. Last but not least, graft is virtually unknown, and such favoritism as exists is mild compared with what flourishes under our American political system. All these things combine to make municipal administration more efficient in Great Britain than in the United States.

Municipal Enterprises Farther Advanced

This greater efficiency favors an extension of municipal enterprises in Great Britain far beyond what is undertaken in America, including entry into various commercial activities. Refuse utilization is one of these, and at Glasgow it is carried to its greatest possible extreme, as the foregoing notes indicate. As is the case with a considerable percentage of British cities, Glasgow owns and operates its electric light and power plant. This gives an intramunicipal outlet for surplus current. In other British cities steam raised by refuse destructors is used by some to pump sewage and, less commonly, water.

The day may come when American municipal conditions will be as favorable to refuse utilization as are British, but it seems far in the future. That the practice has its drawbacks in Great Britain will be noted before closing.

Another fact regarding refuse disposal in Great Britain: Twenty-five years ago few of the cities having what might be called model plants of the period were treating all their refuse in those plants. Instead, varying percentages, sometimes large, were being disposed of otherwise. Taking Great Britain as a whole, the larger part of the city refuse produced was probably dumped on land, with selected portions, in some cases, spread and plowed in. This kind of utilization was practiced perhaps as much to lighten heavy soils as to add crop food.

My impression is that land disposal still accounts for the bulk of the refuse of British cities. Moreover, the Ministry of Health has given general sanction to land disposal by issuing regulations, written after the development of the "sanitary fill" practice at Seattle since the abandonment there of three destructors of one of the leading British types.

I have felt it advisable to point out some of the conditions favorable to refuse utilization in Great Britain that do not hold good in the United States, because from time to time here there is so much well-meant but misleading propaganda to utilize city refuse as it is done abroad, with wild claims of resulting profit, whereas at the best all that is achieved overseas is to reduce part of the cost, if all capital and other charges are considered.

Doubtless the Glasgow destructor will enter into such propaganda. It is too early to say what that plant will show in the way of profit or deficit, for it has not been running long enough under settled working conditions to give even one full year's commercial results as yet. In fact, the contract tests as to fulfillment of contract guarantees had not been completed in July. Capital cost, nearly $3,000,000 on a 540-ton (short) plant (allowing one-fourth held in reserve), will be heavy and operating costs will be high. Whether the electricity output will equal expectations is yet to be determined, although indications thus far are said to be promising. Lack of interdepartmental co-operation, mentioned above, is an obstacle. Very likely it will be overcome, but if it presents difficulties in Great Britain how much worse these might be in America! All the more so because rarely here does one city department get credit for services rendered to another and because even more seldom does our municipal accounting show costs from which proper credits can be deduced.

My intention has not been to question the ultimate success of the Glasgow enterprise, which is the latest word in refuse destructor practice, with heat and other byproducts utilized, but rather to point out that there are still uncertainties as to the Glasgow destructor and that great differences between conditions here and abroad should be given due weight by engineers and city officials when considering whether similar installations should be adopted for use under our widely different American conditions.

Town Water Supply Fully Metered

By S. L. Van Norden
Teaneck, N. J.

IT IS unusual to find a municipality whose water system is 100 per cent metered, but to the borough of Glen Ridge, N. J., belongs this distinction. Due to the fact that a 60-in. steel feeder line connecting the Cedar Grove distributing reservoir with the city of Newark runs through Glen Ridge, it is possible to purchase from Newark an ample supply of water, which is fed by gravity into the cast-iron distributing system of Glen Ridge. The feeder line has been tapped at three points and compound meters have been installed to record the amount of water that Glen Ridge uses.

The water department makes all taps and house connections at the expense of the consumer. Meters are installed by the department free of charge but remain its property. When a house is vacated, the water is turned off at the curb and the meter removed, tested and replaced in stock. In the event that a consumer feels that his meter is registering incorrectly, the department will test the meter at its laboratory. The test consists of allowing water to run through the meter until the dial registers 1 cu.ft. The water is then weighed and the percentage of error determined. It has been found from these tests that less than 1 per cent of the meters tested show an error greater than 2 per cent and, of these, more than half show the error to be in favor of the consumer. The department makes a charge of $2 for testing a meter. If the test shows the meter to be incorrect, allowing a 2 per cent variance in either direction, a new meter is installed and the fee is refunded to the consumer; if, however, the meter is correct, it is replaced in service and the fee is retained. The department is entirely self-supporting, the charges being so computed that the consumer receives water service at cost.

News of the Week

Cleveland Retains City Manager Plan by Small Margin

For the third time within less than two years the electors of Cleveland, Ohio, voted on Aug. 20 to retain the city manager plan of government, the majority against the change being 3,195 in a total vote of 97,155. The previous defeats of attempts to amend the charter were by majorities of 6,416 in November, 1927, and 3,032 in April, 1928, but in 1927 ex-Governor Harry L. Davis, the proponent of change at each of the three elections, had a prominent political leader of his own party against him, while in 1928 there was again a partial split in what would normally have been the Davis ranks. At the present election the split was healed. One of the strong objections to the charter as it now stands is its requirement of proportional representation in the choice of councilmen.

Atlanta Quintuples Its Area

Creation of "Greater Atlanta," Ga., under a borough system of government was accomplished by legislative enactment approved Aug. 20 annexing to Atlanta all of Fulton County except two election districts on the north and, in addition, extending the portion of the city already in DeKalb County. The cities taken in are Avondale, College Park, Decatur, East Point and Hapeville, with combined populations estimated at 50,000. The unincorporated areas annexed are estimated to have another 50,000 population, making the total of Greater Atlanta between 300,000 and 400,000. The area of the city is increased from 36 to 179 square miles.

The act is to take effect Jan. 1, 1930. A council is created composed of mayors and other representatives of the six or more boroughs. This council has advisory powers only, but is to study a plan for future government of Greater Atlanta, using funds to be provided by popular subscription.

Prospective Purchasers of Power From Boulder Dam Meet

Representatives of private interests and municipalities desiring to purchase power from the Boulder dam project met at Denver, Colo., with Elwood Mead, head of the reclamation service, and Raymond F. Walter, chief engineer, to submit proposals. Applicants came principally from four states, California, Arizona, Nevada and Utah. The Southern California Edison Company is the largest private corporation asking for contracts under the Boulder dam.

Dr. Mead's purpose in visiting Denver at this time, he asserted, was merely to examine and analyze propositions submitted for power purchase.

States Improved 29,252 Miles of Highways in 1928

Texas Leads in Year's Improvement, With Kansas Second and Arkansas Third

DURING 1928 the 48 states improved a total of 29,252 miles of their highway systems, according to figures received from the states and compiled by the Bureau of Public Roads of the U. S. Department of Agriculture. This is an increase of 2,530 miles over the 1927 figure of 26,722 miles. The 1928 total includes 8,675 miles of graded and drained roads and 20,577 miles of new surfacing. Of the roads surfaced, 13,843 miles were previously unsurfaced and 3,587 miles were previously improved with a type of surface lower than the one newly applied. The remaining 3,147 miles were previously improved with the same type of surface, and the work done is therefore classed as reconstruction.

The total of 20,577 miles of surfacing placed is classified by types as follows: 1,200 miles of sand-clay and topsoil; 9,623 miles of gravel; 1,006 miles of waterbound macadam; 1,979 miles of bituminous macadam; 225 miles of sheet asphalt; 373 miles of bituminous concrete; 6,055 miles of portland cement concrete; and 116 miles of brick and other block pavements.

Texas heads the states in the year's improvement with 2,356 miles. Kansas, with 1,785 miles, ranks second; Arkansas, with 1,573, is third. Iowa improved 1,345 miles, and Illinois 1,344, ranking fourth and fifth respectively. Minnesota, with 1,243 miles, is sixth; North Dakota, with 1,230, is seventh; New Mexico, with 1,052, is eighth.

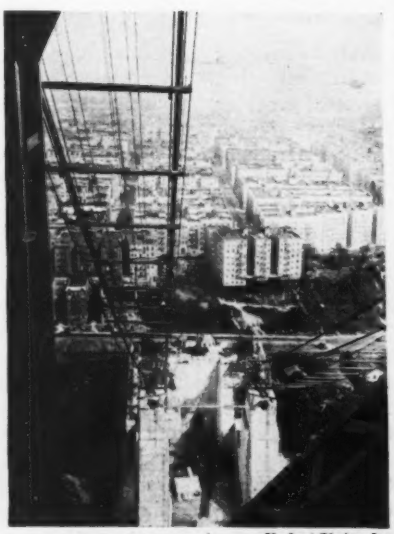
State systems at the end of 1928 had 306,442 miles of highways, according to figures given to the bureau. Of this total, 193,138 miles were surfaced roads, 31,755 miles graded and drained, and 81,549 miles were unimproved and partly graded highways. This is an increase of 13,090 miles over the 1927 total figure. The surfaced mileage is made up of the following types of pavement; 13,499 miles of sand-clay and topsoil; 93,124 miles of gravel; 18,142 miles of waterbound macadam; 15,200 miles of bituminous macadam; 1,498 miles of sheet asphalt; 5,392 miles of bituminous concrete; 42,957 miles of portland cement concrete; and 3,326 miles of brick and block pavements.

Amounts Spent by States

During 1928, the states spent $764,648,134 for state highway and bridge work under the supervision of the state highway departments, made up as follows: for construction and right-of-way, $536,294,303; for maintenance, $158,878,573; for equipment and machinery, $20,505,483; for interest on outstanding bonds and notes, $37,637,034, and for miscellaneous expense, $11,332,741. The states also paid out $27,016,499 in retirement of the principal of outstanding bonds and notes and transferred $35,885,350 to county and town funds for local roads, making a total disbursement of $827,549,983.

For 1928, there was available to the 48 states for state highway and bridge work under supervision of state highway departments a total of $1,035,486,474, made up of an unexpended balance of the previous year's funds of $186,159,876 and a total income of $849,326,598. Of the total income, motor vehicle fees of $259,134,820 and gasoline tax receipts of $234,163,826 allotted to state highways represent more than 58 per cent. Sales of state bonds and notes issued for state highways of $121,483,599 form more than 14 per cent of the total income, while federal-aid fund allotments of $80,798,365 represent more than 9½ per cent. Highway taxes levied in several states amounted to $11,955,782, and appropriations by several states amounted to $42,468,386. Miscellaneous incomes were reported as $12,611,916 and funds transferred from local authorities as $86,709,904.

BUILDING FOOTWALKS ON THE HUDSON RIVER BRIDGE



Herbert Photos, Inc.

Construction of the walkways from which the main cable spinning operations on the Hudson River bridge will be conducted is proceeding rapidly. This view, taken from the New York tower looking toward the New York abutment, gives a good impression of its massive proportions.

Nicaragua to Co-operate in Inter-American Highways

A Congressional resolution approved March 4, 1929, authorized an appropriation of $50,000 to enable the Secretary of State to co-operate with any Latin American government in reconnoissance surveys to develop the facts and to make possible a report to Congress regarding the feasibility, cost and other considerations connected with the construction of an inter-American highway or highways. It was provided that this assistance should be extended to any Latin American government which might make a request or signify a desire for it to the Pan American Union.

The department has received a telegram from Matthew E. Hanna, American Chargé d'Affaires at Managua, saying that the Nicaraguan Minister for Foreign Affairs has formally requested him to inform the department that the Nicaraguan government, in view of the lively interest it has in the construction of an inter-American highway, especially toward the Honduran frontier, desires to accept the benefits of the resolution and requests the co-operation of the United States.

Aluminum Company to Develop Two More Power Projects

The Aluminum Company of America is starting work at Chilhowee, Tenn., and Nantahala, N. C., on the fourth and fifth units of its hydro-electric development of the Little Tennessee River. The first unit, the Cheoah dam, a concrete gravity-type structure 225 ft. high, was completed in 1918, followed by the Santeetlah dam, an arch 200 ft. high with gravity-type abutments, completed last year. The third unit, the Calderwood high-arch dam, is now under construction.

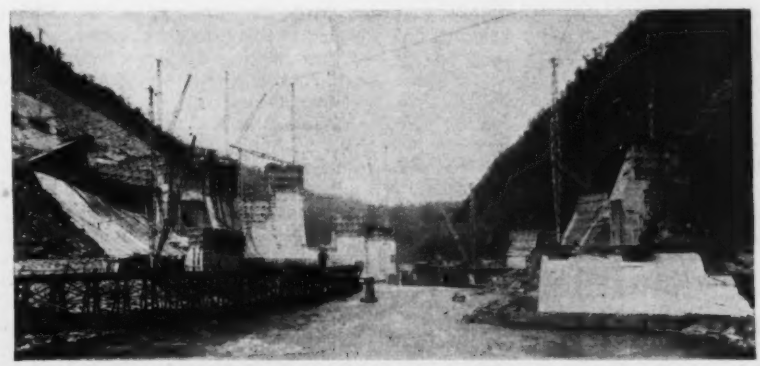
The Chilhowee dam will be a concrete structure of gravity type, 1,500 ft. long and 63 ft. high, situated on the Little Tennessee River about 35 miles south of Knoxville. Headwaters of the Chilhowee reservoir will just reach the Calderwood power house, 10 miles upstream. Flow over the dam will be controlled by twelve Stoney gates 25 ft. wide and 40 ft. high. The power house, located adjacent to the dam on the right bank of the river, will contain two reaction-type turbines of 18,000 hp. each, direct connected to three-phase 60-cycle generators. The power house will contain but one floor, a departure from the usual design. Features of the units will be the use of Kingsbury spherical bearings in place of the flat-plate type, and the introduction of cooling air to the tops of the generators instead of at the bottom.

At Nantahala an arch dam 200 ft. high will be built across a narrow gorge of the Nantahala River, a tributary of the Little Tennessee. There will be no flood-control gates on the dam, the water flowing over the entire crest in a thin sheet, deflected away from the bottom of the structure by jet deflectors. An unlined tunnel, 12x12 ft. in section, will lead from the dam to the power house, located 5 miles downstream. The generating unit will consist of a single reaction turbine of 60,000 hp. operating under a static head of 930 ft.

Work will be begun at once on the Chilhowee dam and shortly on the Nantahala tunnel, which will be driven from one portal and one adit. Construction of the Nantahala dam will not be commenced until the completion of the Chilhowee dam, which is scheduled for Jan. 1, 1931.

Corps of Engineers Creates Railway Section

A railway section of the Corps of Engineers has been organized in charge of Capt. Marshall J. Jones, formerly on duty at the Washington State College. The work of this section is to include the supervision of all railway work assigned to the Corps of Engineers and the preparation of war plans for the construction, operation and maintenance by the Corps of Engineers of railways in the theater of operations. An important part of the activities will be the maintenance of contact with commercial railway systems. The new section will also be expected to give careful consideration to the effectiveness of armored cars and trains and to report on their relative importance in the national defense plan. The collection of data on railways within the United States and the countries for which war plans have been prepared is still another of the tasks that have been delegated to the new section.



CONSTRUCTION VIEW OF THE CALDERWOOD DAM

U. S. to Make Exhaustive Surveys of James River

Flood control of the upper James River, Virginia, will be studied in a survey by U. S. army engineers, $87,000 having been provided by the federal government for the work. Floods in the James River and its tributaries have caused damage to crops and have interrupted shipping, and it is hoped to find a permanent remedy.

In addition an exhaustive survey of the hydro-electric power possibilities of the James River will be begun by the War Department next month under Col. Henry M. Jewett, district engineer at Norfolk. The survey will include a detailed study of all available power sites on the river and its tributaries, the best location for hydro-electric plants, the approximate location of dams, the geologic structure of the river and topography of the stream and its tributaries.

Similar surveys will be made on the Roanoke and Meherin rivers and their tributaries, the Roanoke survey, now under way, to cost approximately $145,000.

Highway Bridge Proposed Across the Firth of Forth

Investigations made under the instructions of the Ministry of Transport in regard to a proposed highway bridge over the Forth, in Scotland, have been summarized in a preliminary report noted in a recent issue of *The Engineer*, London. Of three sites which were studied, the one recommended is about a mile downstream from the Forth railway bridge. The suspension type of bridge is advised on the ground of appearance and economy; the structure proposed would have a 2,400-ft. span, with side spans of 1,040 ft. and a minimum clearance of 150 ft. Alternative plans are submitted for approach highways. The estimated cost for the entire scheme, including land and engineering expenses, is set at from $27,000,000 to $30,000,000, depending upon the highway approaches finally decided upon.

Arkansas to Aid Development of Airways and Airports

Under authority of a recent act of the Arkansas Legislature authorizing the chairman of the Highway Commission, in his discretion, to use the facilities of the Highway Department in developing a system of airways and airports in the state, C. W. Holderbaum, maintenance engineer in the department, has been named as airport engineer. His duties as such will be in addition to those as maintenance engineer.

The Highway Department has granted several requests to lend road machinery and equipment for use in laying out and grading airports in the state. With the appointment of Mr. Holderbaum the department will be able to give greater assistance than heretofore, including engineering advice.

WASHINGTON NOTES

By Paul Wooton
Washington Correspondent

President Favors Transfer of Public Lands—Holding Flood Funds Pending Court Decision

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This plan was first announced in a speech in Boise, Idaho, July 9, by Secretary of the Interior Wilbur. Since that time the Interior Department has been studying the proposed transfer. Mr. Hoover's commitment to the new land policy is viewed as showing that the administration is now ready to close the transaction as soon as Congress passes the necessary legislation. The plan is described in detail in the July 25 issue of *Engineering News-Record*, p. 137.

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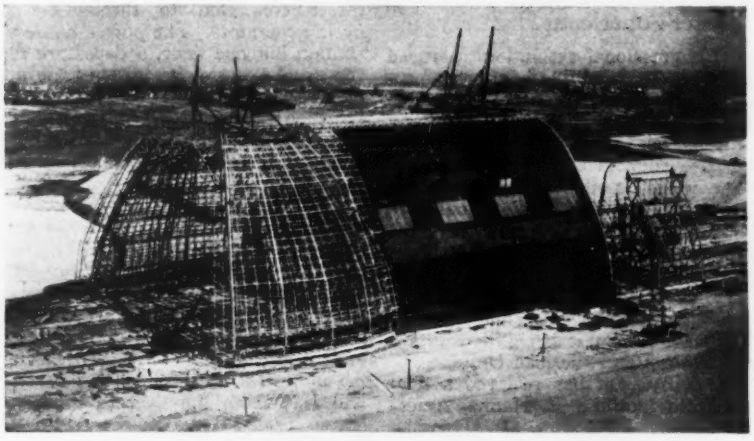

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No Transfer of Flood-Control Funds Contemplated

No reallocation of the $2,000,000 allotted to protection levees in the Boeuf basin will be made until the pending application for injunction in the U. S. District Court for Western Louisiana has been decided. When the Secretary of War stopped work to await the court's ruling, requests were filed immediately for the reallocation of funds to levee and revetment work on the Mississippi River proper. As a decision in the injunction case is expected before the end of the fiscal year, however, the army engineers want to be in a position to proceed with the work on the protection levees in case the injunction is denied. For that reason the fund will be held intact awaiting the determination of the case. There has been some discussion of the possibility that work on other phases of the flood-control program might be held up pending the decision on the floodway injunction. It was suggested that were the west bank diversions eliminated, it would be necessary to increase the capacity of the spillway at Bonnet Carré. It is stated authoritively, however, that work will continue on Bonnet Carré in accordance with the original plan. It is pointed out that the contracts have been let and that plans for the spillway are practically completed. It is also apparent at the Corps of Engineers that there is full confidence that the injunction will not be granted.

Engineers Elected for Golden Gate District

At its monthly meeting on Aug. 15 in San Francisco the directors of the Golden Gate Bridge and Highway District, formed for the purpose of erecting a bridge across the mouth of San Francisco Bay, elected the following board of engineers: J. B. Strauss, president of the Strauss Bascule Bridge Company, Chicago; L. S. Moisseiff, consulting engineer, New York; O. H. Ammann, chief engineer of bridges, Port of New York Authority; and Charles Derleth, Jr., dean of the college of civil engineering, University of California, Berkeley, who served as chief engineer for the bridge built across Carquinez Straits at the upper end of San Pablo Bay in California. Mr. Strauss is to be chief engineer for the Golden Gate bridge.

Committee Named to Study Status of Engineers

With a view to improve the status of the engineering profession, the American Engineering Council announces the appointment of a committee on engineering and allied technical professions, with H. C. Morris, of Washington, D. C., retired mining engineer, as chairman. Other members are: A. B. McDaniels, Washington, representing the American Society of Civil Engineers; Conrad N. Lauer, Philadelphia, the American Society of Mechanical Engineers; H. A. Kidder, New York, the American Institute of Electrical Engineers; L. W. Wallace, Washington, executive secretary of the American Engineering Council.

The committee will collect, tabulate, analyze and disseminate information concerning earnings of engineers. Such information will be so classified as to give a clear conception of the earnings of engineers in the several branches of the profession.

Extensions to Mexico City's Water Supply Planned

American contractors may have an opportunity soon to bid on the construction of additions to the municipal water-supply system of Mexico City which will cost, it is estimated, 15,000,000 pesos, equivalent to about $7,500,000 United States money. Engineers have made a survey of conditions in the capital and federal district, and in their report to President Portes Gil the necessity for improvements is discussed. Among the immediate necessities are the construction of a new 8-mile aqueduct, many miles of additional water mains, an additional pumping plant of large capacity and extension of the distributing system. The ancient aqueduct which runs to the city from Xochimilco is to be repaired. The need of more artesian wells is also pointed out.

Since the beginning of the revolutionary period in 1910 there has been a great influx of people to Mexico City from the rural districts. This increase of population has taxed the capacity of the water-supply system.

Additions to the present drainage system of the valley of Mexico are also contemplated.

Dr. Puig Casaurane is chief of the Water Department of the federal district.

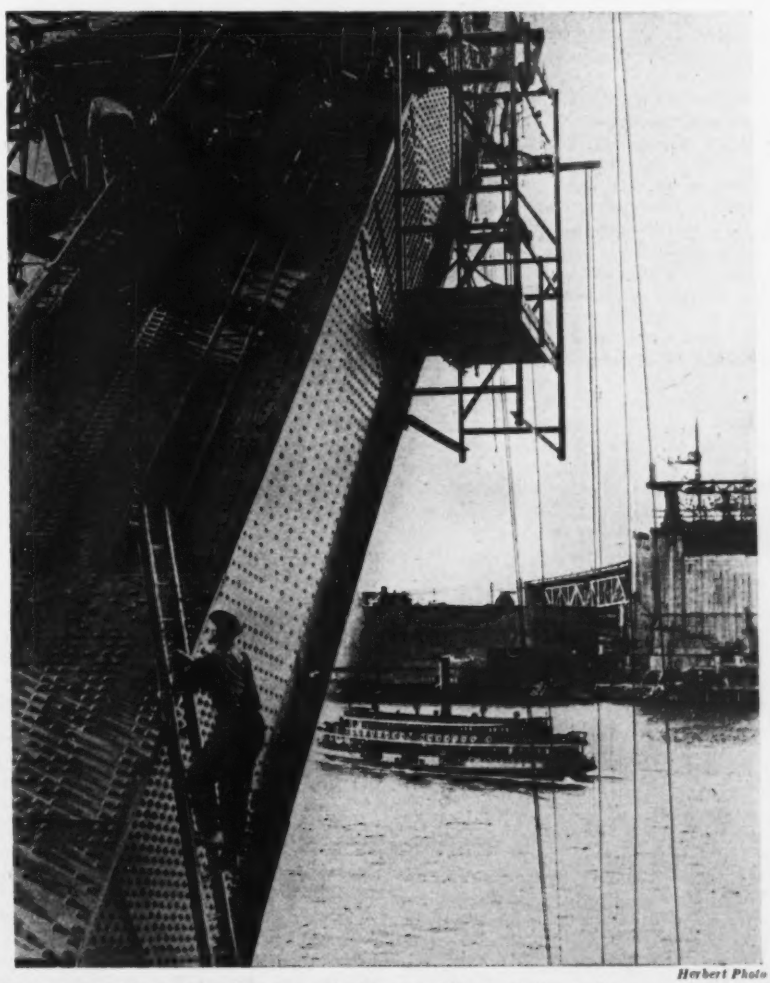
Irrigation Canal, 25 Miles Long, for Rio Grande Valley

Work has begun on an irrigation system to supply water for the reclamation of 129,500 acres of land in Willacy County, Texas. The contract for the dredging work has been awarded to the Trinity Farms Construction Company, Dallas, for $6,000,000. The main canal leading from the 250,000 acre-ft. storage basin will be 130 ft. wide at the top, 90 ft. wide at the base, 14 ft. deep and 25 miles long, and will have a capacity of 250 acre-ft. per hour. In January a bond issue of $7,500,000 was voted for the project. The land to be irrigated is 30 to 50 miles from the Rio Grande, from which the water will be obtained.

One large and two smaller reservoirs will be provided, from which will lead concrete-lined canals and laterals. The engineers' estimate calls for the removal of more than 14,000,000 cu.yd. of earth and the application of 21,000,000 sq.yd. of concrete lining to the canals and laterals. The contract calls for the completion of the project in 900 working days.

The state Board of Water Engineers recently issued a conditional permit for the diversion of 176,000 acre-ft. of flood and storm waters of the Rio Grande for irrigating 88,000 acres of land in Hidalgo County, situated close to the 129,500 acres embraced in the Willacy County district. With the completion of both of these projects, 217,500 acres will be added to the cultivated area of the lower Rio Grande Valley.

BUILDING SYDNEY HARBOR ARCH BRIDGE IN AUSTRALIA



Herbert Photo

Massive steelwork forming part of the 1,650-ft. arch bridge being erected across Sydney harbor, Australia. This photograph was taken as work was being started on the third panel at the south end of the bridge. Detailed information in regard to this structure was published in *Engineering News-Record* in the issue of April 5, 1928, pp. 538-540.

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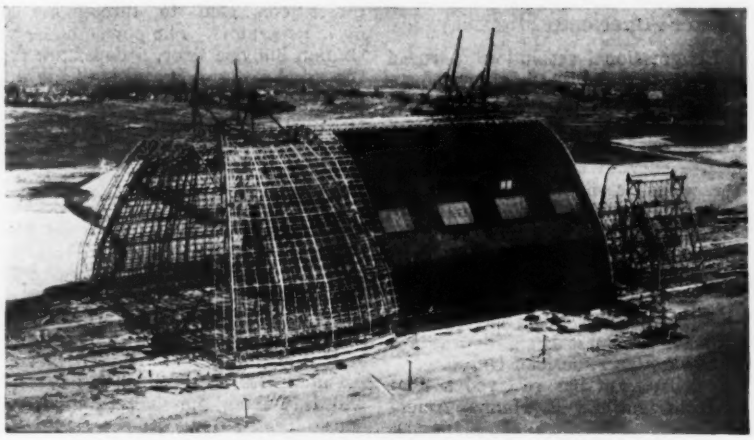

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In practically every application which reaches the commission additional information is sought concerning some of the items making up the pre-license costs. In the Cumberland Falls case Solicitor Russell's opinion was intended only as an expression of view on the facts as he understood them. In the ordinary course, when an item is challenged by a member of the commission's staff, the memorandum is sent to the applicant, with the idea that he will explain it satisfactorily or strike it from the list. There was no intention on the part of the commission to depart from its practice and its own regulations in the case of the much discussed Russell opinions. Mr. Russell explained that he discussed the opinions in confidence with a newspaper man, who promptly violated that confidence and started a controversy which led Senator Couzens to request copies of the opinions. It was he who made them public, and not the commission. It is not the intention of the commission to depart in any way from its usual practice of regarding as confidential the primary discussions with applicants as to items of cost.

No Transfer of Flood-Control Funds Contemplated

No reallocation of the $2,000,000 allotted to protection levees in the Boeuf basin will be made until the pending application for injunction in the U. S. District Court for Western Louisiana has been decided. When the Secretary of War stopped work to await the court's ruling, requests were filed immediately for the reallocation of funds to levee and revetment work on the Mississippi River proper. As a decision in the injunction case is expected before the end of the fiscal year, however, the army engineers want to be in a position to proceed with the work on the protection levees in case the injunction is denied. For that reason the fund will be held intact awaiting the determination of the case. There has been some discussion of the possibility that work on other phases of the flood-control program might be held up pending the decision on the floodway injunction. It was suggested that were the west bank diversions eliminated, it would be necessary to increase the capacity of the spillway at Bonnet Carré. It is stated authoritively, however, that work will continue on Bonnet Carré in accordance with the original plan. It is pointed out that the contracts have been let and that plans for the spillway are practically completed. It is also apparent at the Corps of Engineers that there is full confidence that the injunction will not be granted.

Engineers Elected for Golden Gate District

At its monthly meeting on Aug. 15 in San Francisco the directors of the Golden Gate Bridge and Highway District, formed for the purpose of erecting a bridge across the mouth of San Francisco Bay, elected the following board of engineers: J. B. Strauss, president of the Strauss Bascule Bridge Company, Chicago; L. S. Moisseiff, consulting engineer, New York; O. H. Ammann, chief engineer of bridges, Port of New York Authority; and Charles Derleth, Jr., dean of the college of civil engineering, University of California, Berkeley, who served as chief engineer for the bridge built across Carquinez Straits at the upper end of San Pablo Bay in California. Mr. Strauss is to be chief engineer for the Golden Gate bridge.

Committee Named to Study Status of Engineers

With a view to improve the status of the engineering profession, the American Engineering Council announces the appointment of a committee on engineering and allied technical professions, with H. C. Morris, of Washington, D. C., retired mining engineer, as chairman. Other members are: A. B. McDaniels, Washington, representing the American Society of Civil Engineers; Conrad N. Lauer, Philadelphia, the American Society of Mechanical Engineers; H. A. Kidder, New York, the American Institute of Electrical Engineers; L. W. Wallace, Washington, executive secretary of the American Engineering Council.

The committee will collect, tabulate, analyze and disseminate information concerning earnings of engineers. Such information will be so classified as to give a clear conception of the earnings of engineers in the several branches of the profession.

Extensions to Mexico City's Water Supply Planned

American contractors may have an opportunity soon to bid on the construction of additions to the municipal water-supply system of Mexico City which will cost, it is estimated, 15,000,000 pesos, equivalent to about $7,500,000 United States money. Engineers have made a survey of conditions in the capital and federal district, and in their report to President Portes Gil the necessity for improvements is discussed. Among the immediate necessities are the construction of a new 8-mile aqueduct, many miles of additional water mains, an additional pumping plant of large capacity and extension of the distributing system. The ancient aqueduct which runs to the city from Xochimilco is to be repaired. The need of more artesian wells is also pointed out.

Since the beginning of the revolutionary period in 1910 there has been a great influx of people to Mexico City from the rural districts. This increase of population has taxed the capacity of the water-supply system.

Additions to the present drainage system of the valley of Mexico are also contemplated.

Dr. Puig Casaurane is chief of the Water Department of the federal district.

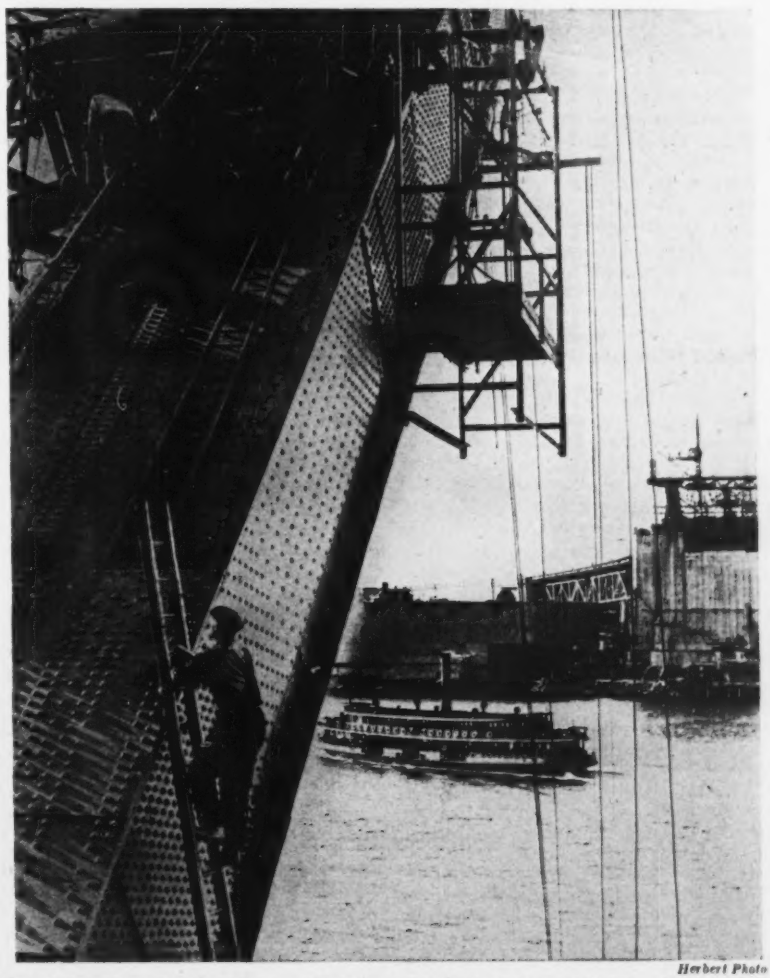
Irrigation Canal, 25 Miles Long, for Rio Grande Valley

Work has begun on an irrigation system to supply water for the reclamation of 129,500 acres of land in Willacy County, Texas. The contract for the dredging work has been awarded to the Trinity Farms Construction Company, Dallas, for $6,000,000. The main canal leading from the 250,000 acre-ft. storage basin will be 130 ft. wide at the top, 90 ft. wide at the base, 14 ft. deep and 25 miles long, and will have a capacity of 250 acre-ft. per hour. In January a bond issue of $7,500,000 was voted for the project. The land to be irrigated is 30 to 50 miles from the Rio Grande, from which the water will be obtained.

One large and two smaller reservoirs will be provided, from which will lead concrete-lined canals and laterals. The engineers' estimate calls for the removal of more than 14,000,000 cu.yd. of earth and the application of 21,000,000 sq.yd. of concrete lining to the canals and laterals. The contract calls for the completion of the project in 900 working days.

The state Board of Water Engineers recently issued a conditional permit for the diversion of 176,000 acre-ft. of flood and storm waters of the Rio Grande for irrigating 88,000 acres of land in Hidalgo County, situated close to the 129,500 acres embraced in the Willacy County district. With the completion of both of these projects, 217,500 acres will be added to the cultivated area of the lower Rio Grande Valley.

BUILDING SYDNEY HARBOR ARCH BRIDGE IN AUSTRALIA



Herbert Photo

Massive steelwork forming part of the 1,650-ft. arch bridge being erected across Sydney harbor, Australia. This photograph was taken as work was being started on the third panel at the south end of the bridge. Detailed information in regard to this structure was published in *Engineering News-Record* in the issue of April 5, 1928, pp. 538-540.

Personal Notes

JULIUS B. MANTHEY has been appointed surveyor and engineer of San Joaquin County, Calif., to succeed Fred E. Smith, deceased.

R. O. SWEEZEY, of Montreal, has been elected president of the Beauharnois Light, Heat & Power Company to fill the vacancy created by the resignation of F. P. Jones.

JOHN H. LAWRENCE, vice-president and manager of Thomas E. Murray, Inc., New York City, since its organization in 1919, has been elected president.

W. D. GRIFFITH, who was previously engaged in flood-control studies with the U. S. Engineers at Cincinnati, Ohio, is now in the engineering department of the Texas Company at Port Arthur.

SAM J. SLATE, formerly Georgia state auditor, who resigned recently, has joined the McDougald Construction Company, of Atlanta, and has been placed in charge of a new department for extension and development work.

JAMES K. JAMES, for six years city engineer and superintendent of streets for the city of Santa Cruz, Calif., is now with the United States Department of Agriculture, Bureau of Public Roads, District No. 2, San Francisco, Calif.

Lieut.-Col. ELLIOT J. DENT, of the Corps of Engineers, has been relieved of duty in the office of the Chief of Engineers and has been ordered to the engineer camp at Fort Humphreys, Va. He will be executive officer of the board on engineer equipment. Colonel Dent was graduated from the army industrial college recently.

WILLIAM H. ALLISON has been appointed assistant professor of civil engineering in Clarkson College, Potsdam, N. Y. After his graduation from Dartmouth College and the Thayer School of Civil Engineering, Mr. Allison was for five years with the Southern and Baltimore & Ohio railroads and afterward was instructor in the Sheffield Scientific School, Yale University.

Lieut.-Col. THOMAS H. EMERSON, secretary of the California Debris Commission, has been relieved of this assignment and ordered to Washington for duty in the office of the chief coordinator. His work with the California Debris Commission will be taken up by Major JOHN R. D. MATHESON, formerly assistant professor of the department of civil and military engineering at West Point.

Major ELROY J. S. IRVINE, who has been assistant to the district engineer of the New Orleans River District, has been ordered to Camp Perry, Ohio, for temporary duty in connection with the rifle matches. On his Mississippi assignment Major Irvine was in charge of the design of the Bonnet Carré spillway. After completing his duty at Camp Perry, he will go to Chicago as assistant to the engineer of the Sixth Corps area.

B. M. DUNCAN, formerly city engineer of Miami, Fla., has been appointed state highway engineer of Florida, to succeed J. L. Cresap, resigned.

OSWALD SPEIR has resigned as hydraulic engineer of the engineering department of the city of San Francisco to become chief designing engineer of the Public Works Engineering Corporation, San Francisco, Calif. This organization handles engineering design and construction for the California Water Service Company, the Oregon Water Service Company and the Washington Water Service Company.

Obituary

ROBERT D. LOCKWOOD, 73 years old, an Indianapolis contractor, died recently as the result of an automobile accident.

DANIEL O. NORTH, 63 years old, well-known Indiana road contractor, died recently at his home in Bluffton, Ind.

HENRY SCUDDER JAUDON, president of the H. S. Jaudon Construction Company, of Elberton, Ga., died in Atlanta recently. He was 58 years old and had been the head of the company bearing his name for a number of years.

THOMAS MCKENZIE, civil engineer and superintendent of the Westerly (R. I.) water-works for more than 30 years, died on July 25. He was well known as a water-works engineer throughout Connecticut and Rhode Island.

TIMOTHY JOSEPH MACCARTHY, formerly city engineer of Springfield and also of Holyoke, Mass., died on Aug. 24 in a New York hospital, where he had gone for a medical examination. He was 67 years old. Mr. MacCarthy was graduated in civil engineering from Queen's College, Cork, Ireland.

JAMES R. HENRY, 79 years old, railroad builder and superintendent of construction of the Chicago-New York Air Line, an electric road built in Laporte, Porter and Lake counties, Indiana, in 1910, is dead at Valparaiso, Ind. He was born in Rochester, N. Y., and when a young man was employed by the Michigan Central Railroad. He later took up railroad contracting work and assisted in building the Chicago & Erie, the Michigan & Ohio and the Rock Island extension in Kansas.

WILLIAM C. PULSE, 70 years old, for years one of the most prominent general contractors in southern Indiana, died recently at Joliet, Ill., following a stroke of paralysis. As a member of Pulse & Porter, he erected the southern Indiana hospital for the insane at Madison, Ind.

THOMAS J. FRAZIER, pioneer railroad builder, died Aug. 19 in Tiffin, Ohio, at the age of 80 years. Mr. Frazier, after his graduation from the Virginia Military Institute in 1871, entered the employ of the Baltimore & Ohio Railroad and later was chief of the engineering corps which surveyed the Chicago division of that road, and as resident engineer supervised its construction. Before retiring fourteen years ago, he served as chief of the maintenance of way, real estate and tax departments of the Chicago division.

ROBERT H. ANDERSON died suddenly at Blue Ridge, Ga., on Aug. 8, at the age of 52. Mr. Anderson was one of the first engineers on the Panama Canal. He served on the United States Deep Water Survey in New York and later as engineer on the investigation of New York City water supply. From 1900 to 1901 he served on the Nicaragua Canal surveys. He was with the United States Geological Survey from 1903 to 1904, when he was appointed assistant engineer for the Isthmian Canal Commission. From 1905 to 1909 he was engineer for the McCall's Ferry Power Company on the Holtwood power dam. In 1909 he became connected with the Tennessee Electric Power Company, being resident engineer for the J. G. White Construction Company on the construction of two hydro-electric developments for the Tennessee Power Company at Parksville and Caney Creek, on the Ocoee River. From 1915 to 1917 he was located at Rock Island, Tenn., in charge of construction of the Great Falls hydro-electric development for the same company. From 1919 to 1921 he was engineer for the J. G. White Construction Company, located at Rifton, N. Y., during the building of the dam and power plant on the Wallkill River. He also did other engineering work for the J. G. White Construction Company at Langley Field, Va., and at the U. S. nitrate plant at Muscle Shoals. In 1922 he returned to the Tennessee Electric Power Company as hydraulic engineer. During 1923 and 1924 he was in charge of construction of additions to the dam and power house at the Great Falls plant. He was in charge of the construction of an earth storage dam on the Toccoa River for the Toccoa Electric Power Company, a subsidiary of the Tennessee Electric Power Company, at the time of his death.

Society Calender

AMERICAN WELDING SOCIETY, New York; fall meeting, Cleveland, Ohio, Sept. 9-13.

NEW ENGLAND WATER WORKS ASSOCIATION, Boston; annual convention, Portland, Me., Sept. 16-18.

AMERICAN SOCIETY OF CIVIL ENGINEERS, New York; fall meeting, Boston, Mass., Oct. 9-11.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION, Chicago; annual meeting, New Orleans, La., Oct. 15-17.

ASPHALT PAVING CONFERENCE, New York City; annual meeting, West Baden, Ind., Oct. 28-Nov. 1.

Construction Equipment and Materials

Injunction Issued in Chlorinator Patent Dispute

An injunction restraining the Paradon Manufacturing Company, Arlington, N. J., from making, using and selling chlorinating apparatus using a means of mixing the chlorine with a minor flow, which apparatus, it is claimed, infringes patents owned by Wallace & Tiernan Company, Inc., of Newark, N. J., has been issued by the U. S. District Court for the District of New Jersey. The injunction will be in effect until the settlement of a pending suit for infringement, unless previously dissolved by the court.

New Developments

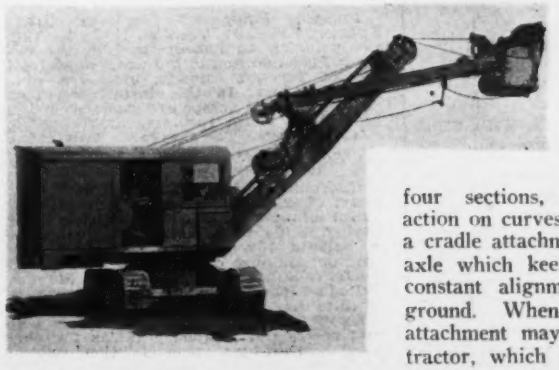
Gasoline Shovel Improvements and New Models Announced

Recent refinements in the shovels, cranes and draglines manufactured by the Northwest Engineering Company, Chicago, Ill., include a new close-quarter crowding device and power shifting of the heavy operating clutches. The close-quarter crowding device does not replace the standard cable crowd of Northwest machines but supplements it. The close-quarter crowd is used only when forcing a bucket well beyond the end of the boom. As in the standard cable crowd, none of the engine power is divided between crowding and hoisting, since by a special reeving of the hoisting cable the dipper is thrust into the bank at the same time it is raised by the hoist cable. A variation in the crowding ratio is obtaned by a slight change in the reeving of the cable.

The "feather-touch" clutch control utilizes the force of rotation of the drum to throw the clutches, the operation being controlled by means of an improved bell-crank linking. The action of the new control is in direct ratio to the pressure on the operator's lever and the feel of the load is always present.

The company has announced a model 6 machine which, as a shovel, has a capacity of 1½ cu.yd. and is convertible to a 19-ton crane, a 1½-cu.yd. dragline or a 49-50-in. trench pull-shovel. This follows closely the design used in previous Northwest machines and can be equipped, if desired, with the new crowding mechanism and with a variable speed motor, accelerator controlled, in place of the regular governor-controlled motor.

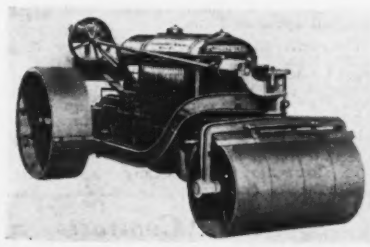
Another recent addition to the Northwest line is the model 7, designed particularly for drainage, sewer and irrigation work. This is equipped with a 1¾-cu.yd. dragline bucket and a 50-ft. boom, but is convertible to a crane or trench pull-shovel. This machine is mounted on a crawler base 11 ft. 6 in. wide and 16 ft. 4 in. long, and has the variable speed motor as standard equipment. Model 7 is also built as a shovel.



NEW 1¼-CU.YD. GASOLINE SHOVEL

Roller Attachment for Tractors

Light rollers built in three weights, 3¾, 4½ and 5¼ tons, are now being produced by the Davenport Locomotive & Manufacturing Corporation, Davenport, Iowa. These may be obtained complete with power plants, or frame and rollers may be purchased separately and attached to Fordson, McCormick-Deering or Massey-Harris tractors. A side cranking device is provided for starting the tractor motor and steering is made easy by a worm and segment steering gear and a division of the front roller into four sections, giving a differential action on curves. The roller frame has a cradle attachment to the tractor rear axle which keeps the power plant in constant alignment even over uneven ground. When not in use the roller attachment may be removed from the tractor, which can be used for other purposes.



LIGHT ROLLER WITH TRACTOR POWER

Joint for Concrete Roads Uses Air as Cushion

Settlement of incompressible material into expansion joints while open is eliminated by the use of an air cushion expansion joint recently developed by the Truscon Steel Company, Youngstown, Ohio. This consists of metallic sides forming a free air space closed at the top by a bellows-shaped piece of galvanized copper-bearing steel which prevents dirt from working into the joint from the surface of the pavement. It is made in lengths up to 6 ft. Two types are provided, one with a continuous dowel plate which permits necessary expansion and contraction while keying together adjacent slabs; the other without this feature, but punched to receive the usual dowel bars. The top of the joint is set below the surface of the concrete, so that spreading, rolling, floating or belting operations can be carried on in a normal manner. Afterward the top groove is cleaned out and edged, and the space, if desired, filled with bituminous joint filler. The manufacturers state that this joint will close almost 100 per cent of its thickness with practically no resistance.

AIR-CUSHIONED EXPANSION JOINT WITH CONTINUOUS DOWEL PLATE

Steel Roof Has Smooth Surface

Three products in the steel roofing field have been announced by the Genfire Steel Company, Youngstown, Ohio. The first of these is known as Rigideck, and consists of 6-in. widths of Armco iron, each section being formed by the pressing of a self-locking rib section into a strip of the iron. This rib is an I-section and interlocks with adjacent sections, providing rigidity and bearing points to rest on the purlins. The roof presents a smooth surface to receive the insulation and can be erected without perforating the steel.

Other types of roofing manufactured by the company are known as Ferrodeck

INTERLOCKING STRUCTURE OF STEEL ROOF DECK

and I-Plates. The former is recommended for the replacing of wooden and other roofs on old buildings. The latter comes in rigid 2-ft. sections, with ribs at 6-in. intervals.

Side Stability Increased by Auxiliary Crawler Treads

Outrigger crawlers, in addition to the usual traction treads, are used in the new 2-cu.yd., 60-ft. boom dragline produced by the Page Engineering Company, Chicago, Ill. The result is unusual stability, giving a security against overturning in a lateral direction practically equal to that lengthwise of the machine. In shipping, the auxiliary treads are removed, permitting the machine to be loaded upon a flat car without dismantling the traction or drum treads. The machine, which may be rigged as either dragline or crane, swings on an 11-ft. rail circle and has the boom sockets an unusually large distance from the center pin, giving a range with a 60-ft. boom equal to that

AUXILIARY TREAD GIVES SIDE SUPPORT TO DRAGLINE CRANE

of many 65-ft. boom machines. Frictions for swinging are as large as those used for loading or hoisting and are controlled by compressed air. The machinery deck is similar in arrangement to other Page machines, with plenty of room for repairs. Gear drive is used for the traction unit. Power is provided by a Page diesel engine.

Business Notes

PORT STOCKTON CEMENT COMPANY, Stockton, Calif., has been formed. Plans for the construction of a new plant for the manufacture of cement are being prepared. Officers of the company are: William A. Sherman, president; W. C. Stevenson, chief engineer; and Alfred E. Roberts, consulting engineer and designer.

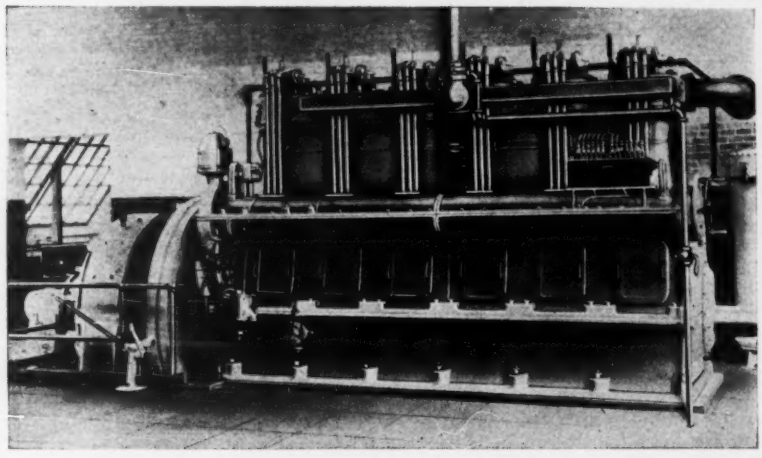
WESTINGHOUSE ELECTRIC INTERNATIONAL COMPANY has appointed I. F. Baker European sales manager with headquarters in London, England. Mr. Baker was formerly sales manager of the power division of the company in New York City.

SMITH ENGINEERING WORKS, Milwaukee, Wis., has awarded the contract for the construction of a new fireproof building 150x90 ft. to increase its manufacturing facilities.

GEORGE D. WHITCOMB COMPANY, Rochelle, Ill., has started construction of a new factory unit to be used primarily for the erection of large oil-electric locomotives.

AMERICAN LAFRANCE & FOAMITE CORPORATION, Elmira, N. Y., is transferring its Utica, N. Y., office and plant to Elmira.

ORTON CRANE & SHOVEL COMPANY, Chicago, Ill., has appointed Thomas D. Crowley & Company, Chicago, its special railway sales representative in the United States.



LARGE INTERNAL-COMBUSTION ENGINE USES GAS AS FUEL

Certain changes in the diesel engines for stationary use built by the Chicago Pneumatic Tool Company, New York City, have resulted in the production of similar engines of the vertical heavy-duty type using gas as a fuel, varying from blast furnace gas to natural gas of high heat content. These gas engines are produced in three-, four- and six-cylinder models, developing 225, 300 and 450 hp. at 257 revolutions per minute. A feature of the engines particularly useful where the gas supply is uncertain is that they can easily be converted to solid injection, full diesel engines operating on fuel oil. This permits temporary operation on gas without incurring the risk of future high fuel costs or heavy replacement expense.

New Publications

Pumps—BYRON-JACKSON COMPANY, Berkeley, Calif., has issued bulletin 295 describing its type S double-suction single-stage centrifugal pumps with horizontally split case.

Pavers—FOOTE COMPANY, Nunda, N. Y., has issued a new 34-p. illustrated catalog describing Multi-Foote pavers and giving detailed specifications.

Concrete Coating—THE BARRETT COMPANY, New York City, has issued an illustrated booklet describing the results to be expected and the method of application of Protectar, a tarry compound in liquid form for the protection of concrete surfaces from disintegration.

Arc-Welding Instruction—GENERAL ELECTRIC COMPANY, Schenectady, N. Y., has issued an 8-p. illustrated booklet describing the free school maintained by the company at Schenectady for the training of dependable arc welders.

Road Construction—CONCRETE STEEL COMPANY, New York City, has issued a 32-p. illustrated booklet "Economical Concrete Road Reinforcing," describing in detail modern methods of concrete highway reinforcement and the method of laying a concrete road.

Excavating Equipment—PAGE ENGINEERING COMPANY, Chicago, Ill., has issued new bulletins describing the several types of buckets, dragslips, slackline cableways and drag shovel attachments manufactured by the company.

Chlorinators—PARADON MANUFACTURING COMPANY, Arlington, N. J., in technical bulletin 21 gives operating data and descriptions of Paradon dry feed chlorinators.

Valves—HOMESTEAD VALVE MANUFACTURING COMPANY, Coraopolis, Pa., lists and describes all valves manufactured by the company in a new 52-p. illustrated catalog.

Zinc—NEW JERSEY ZINC COMPANY, New York City, in a 44-p. notebook-size booklet discusses the physical properties, manufacture, treatment and use of rolled zinc.

Deep Well Pumps—WORTHINGTON PUMP & MACHINERY CORPORATION, Harrison, N. J., in bulletin D-450 B-1, describes in detail the construction and installation of Axiflo, Hiflo and Coniflo deep-well pumps and accessories. Included in the bulletin are several hydraulic tables and a page of instructions for determining the alignment of a deep well.

Pumping Equipment—DE LAVAL STEAM TURBINE COMPANY, Trenton, N. J., describes the pumping equipment used in the new $14,000,000 addition to the water-works system of Toronto, Canada, in a 4-p. illustrated leaflet. In this installation fourteen centrifugal pumps are used; six driven by geared steam turbines, the others by electric motors. The aggregate daily capacity of the pumps is approximately 325,000,000 Imperial gallons.

Hydraulic Data—CHICAGO PUMP COMPANY, Chicago, Ill., gives sixteen pages of useful information and miscellaneous tables having to do with pumps and hydraulics in bulletin 150. Among subjects treated are: determining capacities of pumps and ejectors; atmospheric pressure at different altitudes; theoretical horsepower required to raise water to different heights; quantities of feedwater required to supply boilers; speed and frequency tables for electrical apparatus; pipe friction tables; discharge through various size orifices under pressure; and instructions for operating centrifugal pumps.

Costs and Contracts

E. N.-R. Index Numbers

Cost			Volume		
Aug. 1,	**1929**	**205.81**	**July,**	**1929**	**320**
July, 1,	1929	204.77	June,	1929	347
Aug. 1,	1928	207.29	July,	1928	266
Average,	1928	206.78	Average,	1928	287
Average,	1927	206.24	Average,	1927	263
1913.....		100.00	1913.....		100

This Week's Contracts

Heavy construction contracts, reported by *Engineering News-Record* in the week of Aug. 29, with some comparisons, total as follows:

(In Thousands of Dollars)

	Aug. 29, 1929	Aug. 22, 1929	Aug. 30, 1928
Building:			
Industrial.....	$4,355	$12,565	$4,874
Commercial...	29,594	78,218	21,668
Streets and roads.	13,261	15,323	16,726
Other eng. constr.	9,949	9,058	16,871
Total.....	$57,159	$115,164	$60,139
Total, all classes, Jan. 1 to Aug. 29:			
1929.....			$2,895,657
1928.....			2,397,614